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BED-LOAD TRANSPORTATION IN MOUNTAIN CREEK

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FOREWORD

As this global war progresses, it becomes more and more evident that production of food and other agricultural products is of foremost importance to Victory and to the Peace that is to follow. All available land of high fertility should be used for crop production and, through good farm management and soil and water conservation, made to produce as intensively as possible. The amount of this class of land is large, but, when compared to the expected need for crops, is not large enough. Quality of land is now more important than quantity, because it can produce more with less manpower, equipment, and fertilizer. Bottomlands and alluvial soils adjacent to streams, if free from frequent flooding, and well drained, are normally the most fertile of all lands. But, in many places throughout the United States, such valley lands cannot now be used to their fullest extent, or at all, because of sedimentation, which obstructs drainage, increases flooding, and impairs soil fertility by overwash of the infertile residue of soil erosion.

The research work of the Soil Conservation Service has indicated that sediment-control measures, if properly applied, would prevent a large part of this damage, would reclaim land already damaged, and would protect irrigation and drainage developments. Methods of sediment control have not yet been as fully developed, however, as many other soil and water conservation measures. Remedies for the sedimentation problem in each valley still require individual study and treatment.

This publication presents the results of experiments on sediment transportation in Mountain Creek, a typical small stream of the Southern Piedmont. These experiments have developed methods for collecting data needed in the improvement of small valleys for crop production. They have resulted in development of methods (1) for estimating the sediment load, particularly the heretofore unmeasurable bed load, of small streams, (2) for estimating the effects, in small valleys, of reducing sediment loads through conservation practices, and (3) for estimating the effects of channel improvements, such as cutoffs, on flooding and channel sedimentation.

INTRODUCTION

The deposition of sediment in stream and river channels has caused much damage in many parts of the United States and the alleviation or prevention of such damage where it occurs is an important objective of any conservation or flood-control program. Unfortunately, little is known concerning the principles that govern the transportation and deposition of such injurious sediment, and what is known has been derived from small scale laboratory studies. The applicability of such laboratory investigations to natural streams has not been verified and, consequently, plans for remedial work have been based on "reasonable" assumptions rather than on scientific principles.

The most common practical problems that are involved in the study of stream channel sedimentation are of three general types. First, if the amount of sediment delivered to a stream channel is reduced by erosion-control practices or other measures, will the channel capacity be increased by scouring and, if so, how much? This has been one of the most important unsolved problems on flood-control surveys, where the benefits from proposed remedial programs must be evaluated in advance of their application. Second, if the present rates of sediment contributions remain the same or increase in the future, what will happen to the stream channels? Third, in places where new channels are to be dug, or the present channels modified by cut-offs, bank clearing, etc., what will be the effect of the new conditions on the capacity of the channel to transport its sediment load, to degrade its bed, or to modify its shape by lateral erosion? Because all three of these problems require knowledge about the available sediment supply and the capacity of the channel to transport it, they are basically similar.

In aggrading channels, the injurious sediment is usually sand or coarser debris that is transported in large part by rolling, sliding, or bouncing along the stream bed. For convenience this material usually is called "bed load." From laboratory investigations, formulas have been developed to describe the movement of bed load under varying experimental conditions, but the applicability of these formulas to natural streams where the conditions of sediment transportation are more complex has not been verified. The purpose of this report is to give the results of bed-load measurements in a natural stream and to show that certain formulas are applicable to natural streams and that they can be used to great practical advantage in solving the three types of problems outlined above.

The term "bed load" has been used and defined in the literature in several different ways (10)[1/]. In the solution of practical problems

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1/ Numbers in parentheses refer to Literature Cited p.

of stream-channel equilibrium, only that part of the sediment load need be considered that can take part in changes in the conformation of the bed by deposition or scour. This part of the moving sediment, at least temporarily, has been and again will be part of the stream bed. This material, for the purpose of this discussion, is considered as "bed load" and defined as "bed material in movement." According to this definition, the net deposition or scour in a reach of a river is the difference between the bed-load transportation of the upper and lower ends of the reach.

The general problem of predicting bed-load transportation on a certain bed as a function of the flow has been approached simultaneously from two different sides; first by actual measurements in natural streams, especially in the Enoree River (1), and second, by an analysis of the results of all available published and unpublished flume experiments (7). The latter work was done with the intention of checking empirical transportation laws derived from flume experiments against river conditions, thus eliminating from further consideration those formulas that could not be extrapolated to river scale. This method has proved very efficient in similar studies in Switzerland, where the transportation of coarse bed load in mountain streams was related to flume studies by partly known and partly newly developed principles of similitude. Unfortunately, these laws of similitude can not be applied to streams like the Enoree River where the sediment is so fine that the further reduction in size necessary in any flume study would introduce new factors of unknown effect.

It was concluded, therefore, that the problem of the transportation of relatively fine bed load could be approached best in a series of steps involving both field observations and the use of certain principles established by flume studies. It was further concluded that the first of such steps would be measurements on a natural stream with the same or a similar sediment mixture as the Enoree River but with a water depth and with flow conditions comparable to those usually prevailing in a flume. Mountain Creek, a tributary of Enoree River with a drainage area of 11.7 square miles, seemed to possess these characteristics, at least for its lower stages.

Mountain Creek is a typical small stream similar in general character to hundreds of small streams throughout the Piedmont Region. Because of its lack of special features and because it is in equilibrium, or essentially so, Mountain Creek was considered well suited for use in this study. Equilibrium is especially important, because duplicate measurements are not possible where the composition and position of the bed change rapidly.

This study, in addition to its importance as a stepping stone in the solution of the problem on larger rivers, will prove of practical value in its application to other similar small Piedmont streams. The

principles as developed permit the determination of the transportation capacities at certain cross sections on which depend the whole fate of the streams and of the valuable bottom lands along these streams. An inexpensive determination of this capacity and its change due to proposed changes in the river is especially important because the values involved are generally too small to warrant expensive investigation. Under the heading of application, the method of applying the principles to small streams is demonstrated.

A great number of observations on sediment transportation indicates that bed load moves slowly, even very slowly, compared to the water and the suspended load. Any changes in the sediment supply in the watershed, therefore, will affect the bed-load movement only after a relatively long period of time. Short time fluctuations in this supply may occur but they are soon smoothed out by temporary deposit and scour in the bed. For this reason, the momentary rate of bed-load transportation at any one point of the river can be expressed best as a function of the local hydraulic and bed conditions rather than of the sediment supply. In the long run, however, the fate of the river sections and of its profile will be determined by the sediment supply. By means of local and temporary deposit and scour the hydraulic conditions in a particular reach of the stream are gradually changed until it is able to transport the average sand supply. Unfortunately, any change in the river profile due to changes in sediment supply often is detrimental to the agricultural use of the bottom lands. For this reason, engineers usually have tried to prevent changes of the profile of rivers by changing its cross sections.

The best hope of success in planning corrective measures, however, lies in giving proper consideration to the laws of sediment transportation. It is hoped that the following description of the Mountain Creek study and the results obtained will contribute to the general knowledge of the application of these laws.

The present investigation was made as part of the work of the Greenville Sediment Load Laboratory under the direction of G. C. Dobson, Project Supervisor. The help in editing the manuscript given by Mr. Dobson and Dr. G. Rittenhouse is greatly appreciated. The writer is also very grateful to his colleagues J. W. Johnson and A. G. Anderson for taking over certain special phases of the work which could be separated from the main problem. The assistance of Mr. A. T. Talley in the construction of the apparatus and in conducting the field measurements was particularly helpful.

DESCRIPTION OF MOUNTAIN CREEK

Mountain Creek is a tributary of Enoree River. Its drainage area of 11.7 square miles lies entirely in Greenville County of South Carolina and includes part of the east side of Paris Mountain, the highest part of which is 2,047 feet above sea level.

Land use on the drainage area consists of 58 percent woodland, 28 percent cultivated and mostly in cotton, and the remaining 14 percent abandoned and urban land. The soils in the uplands range mostly between Cecil sandy loam and sandy clay loams, while Porters loam is predominant in the highest part of the watershed on Paris Mountain.

Erosion is rather well under control in the watershed, especially on Paris Mountain where part of the watershed is included in Paris Mountain State Park.

At the junction of Mountain Creek and Enoree River the elevation of low water is about 820 feet. About 0.7 mile above the junction an easily accessible section was chosen for making observations on bed-load transportation. The corresponding hydraulic measurements, such as the slope of bed and water surfaces, the average cross section, the water velocity, were made in a 750-foot reach above the measuring section.

The cross section and profile within this reach are rather regular with an average width of the sand bed of 14.22 feet. The banks are rather steep and were assumed as an average to have a slope of 45°. During the measuring period the banks were heavily covered with grass and other seasonal vegetation, as may be seen in Figure 1. The tops of the banks are between 2 and 3 feet above the average bed. The flood plain in this part of the valley is 300-500 feet wide, and regularly planted in corn.

The climatic records of the Weather Bureau station at Greenville are fairly representative for the watershed. They are as follows: The mean annual temperature is 59.1° F. The mean temperature for the winter months is 41.2° F. with a minimum of -5° F. and a maximum of 82° F. The snowfall is usually light and seldom stays on the ground more than 2 or 3 days. The mean summer temperature is 75.7° F. with a minimum of 40° F. and a maximum of 106° F. The mean annual rainfall is 51.96 inches, with a total of 42.66 inches for the driest year (1899) and 77.83 inches for the wettest year (1901). The average from April to September, inclusive, is 28.88 inches.

Below the measuring section, the creek flows through a narrow gorge-like valley where the bed contains a great amount of coarse particles originating in an old lateral gully. This reach controls the stage at the measuring section. In this narrow valley the banks are

lined with trees and thick brush. Trees and brush were cleared away from the banks of the measuring reach some 10 years ago and it has been so maintained ever since.

The clearing ends about 800 feet above the measuring section and from there on up stream the banks are again wooded. Approximately 0.9 mile above the measuring section a small dam backs up the water in a small reservoir which is largely filled with sediment. Only a small channel is kept open in the reservoir by periodically flushing through a sluice gate in the dam. When the sluice gate is open, all the sediment transported by the stream passes through the reservoir. The present capacity of the reservoir is about 200,000 cubic feet or 4 1/2 acre-feet. This volume of water can be flushed through the sluice gate in about 1 hour. This flushing cleans out the channel in the reservoir and deposits most of the sand in large bars below the dam from where it is removed during the period of normal flow.

Twice during the measuring period these flushing discharges were used to obtain measurements for higher flows than would otherwise have occurred. These artificial floods of duration of about 1 hour have a distinctly different hydrograph than a natural flood. The most important difference seems to be that the discharge decreases very rapidly in the latter part of the flood, while after all natural floods the discharge recedes over a number of hours in returning to the normal flow. This difference has a distinct influence on the flow characteristics towards the end of the flood and will be described later.

No significant tributary enters the creek between the dam and the measuring section. No signs of gullying or severe bank erosion could be found. Therefore, sand transportation and discharge are constant over the whole stretch. Within the measuring reach itself, the steep banks do not seem to be sufficiently protected as some bank erosion with caving can be observed. The amount of this eroded material seems to be small compared with the normal bed load and, therefore, has not been taken in consideration in this study.

APPARATUS FOR THE MEASUREMENT OF BED LOAD

So far as known, prior to this work, no satisfactory movable instrument has ever been used to measure the bed load in small natural streams. The instruments used in all previous attempts to measure the rate of bed-load transportation can be grouped under the name "bed-load traps." These bed-load traps, described in a recent publication (14) consist primarily of a box or pan-like container which is lowered to the bed and allowed to collect material for a certain length of time. The flowing water enters the instrument and carries the bed load into the container. Water and sediment are separated by means of screens or by a sudden retardation of the water. The water then leaves the instrument

while the sediment is trapped, thus indicating the amount of bed load for a certain period of time and certain width of the stream. Unfortunately, this rather convenient method of measurement is not very satisfactory. Especially fine, sandy bed load is very difficult to measure because, if the influence of local fluctuations in the rate of transportation is to be eliminated, the measurement must consist of an extremely large number of single determinations even under the most favorable conditions. In small streams the duration of floods is relatively short and, consequently, there is not sufficient time to permit the determination of bed-load transportation with a trap. It was concluded, therefore, that any method of measurement, to be satisfactory, must directly measure the total amount of the transported bed load. This is the principle underlying the design of the Enoree River Sediment Load Laboratory (1). Based on experience obtained at this laboratory, a portable apparatus was developed to accomplish this purpose.

The hopper

This apparatus is intended for the measurement of that part of the bed material in movement that cannot be measured as suspended load because it moves close to the bed where its velocity cannot be determined. This bed material is continuously making contact with the bed and often interrupts its movement by periods of rest. The average distance between two points of rest has been measured to be about 100 times the diameter of the grain (3). For an average diameter of the grain in Mountain Creek of 0.9 mm this average distance would be 90 mm or less than 4 inches. As the probability for longer steps decreases logarithmicaly, a step of 2 feet has only a probability of 2^{-9} or about 1 in 500. Therefore, if a slot is made to extend completely across the stream bed 2 feet wide in the direction of flow, 99.8% of this part of the bed load will be deposited in it. Any bed material moving in suspension must be determined separately by means of suspended-load samples. In the case of Mountain Creek suspended-load samples showed that no significant amount of bed material moved in suspension during the observed stages. A two-foot slot, therefore, would catch essentially all of the bed load as defined in the "Introduction."

The hopper as shown in Figure 2 is the mechanical realization of this slot. It consists of a welded sheet-iron box with a 1-inch screen covering the open top to exclude any coarse trash. This hopper is lowered into the sandy stream bed until the top is several inches below the lowest parts of the bed. On the upstream side where the sediment approaches the hopper it assumes its natural angle of repose from the bed down to the edge of the opening. All metal walls of the hopper are steeper than this angle and, therefore, all sand will slide down to the lowest point in the hopper. The sand-water mixture is pumped continuously from the hopper into the separator-weighing tank.

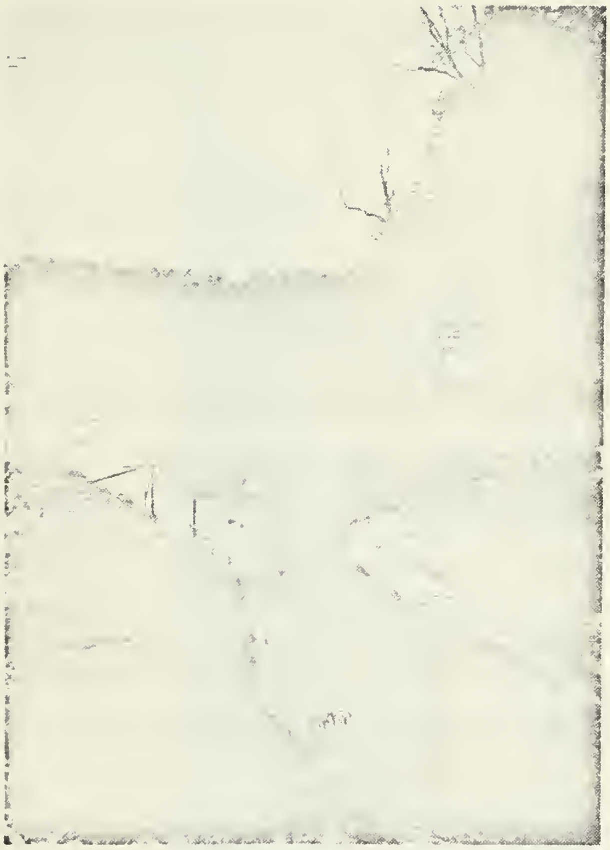

Figure 1.-- Measuring device at Mountain Creek

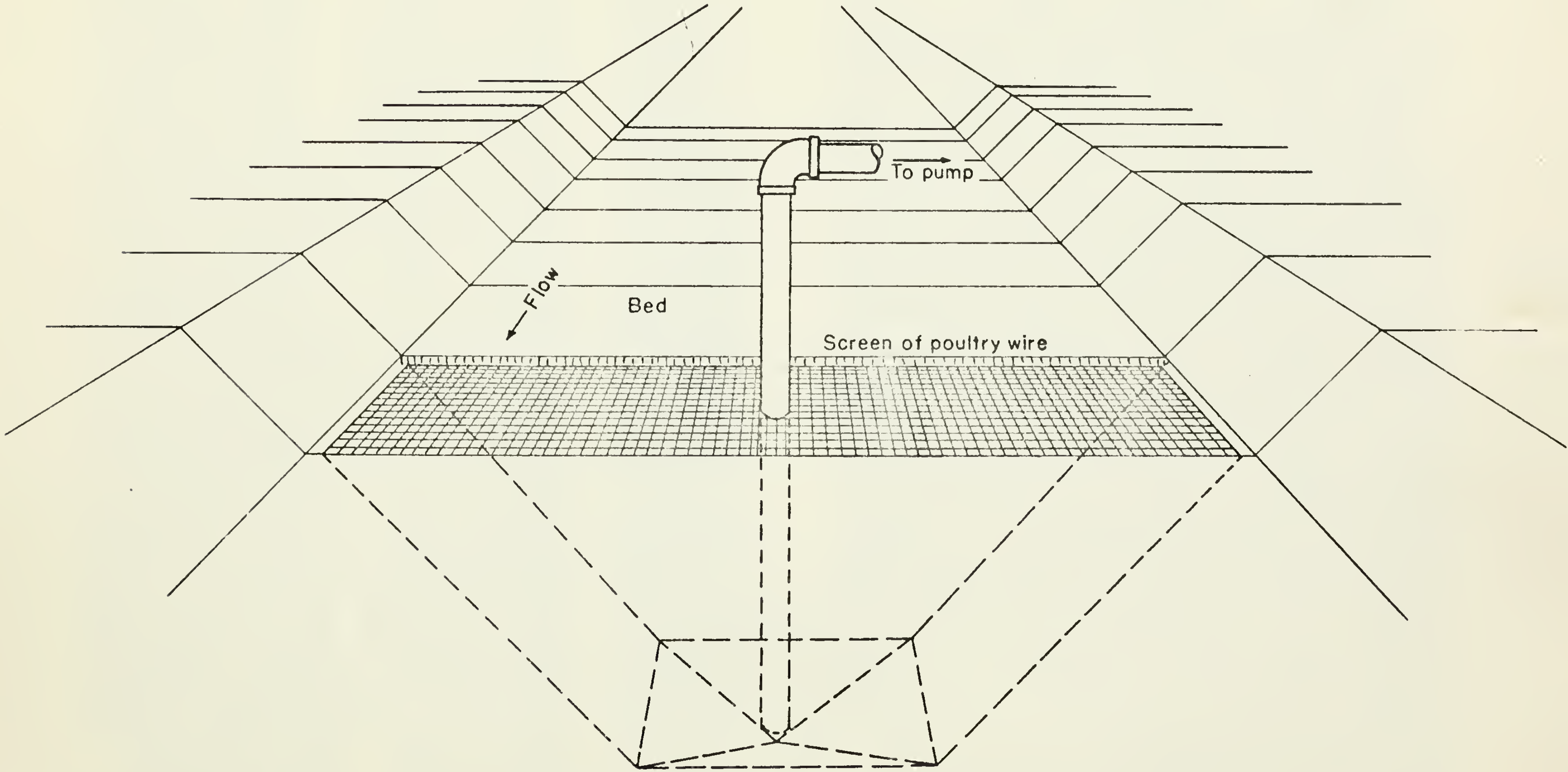

Figure 2 – Diagrammatic view of hopper located in the stream bed.

This hopper and the 3-inch suction pipe are the only parts of the whole mechanism that must remain in place for the complete duration of the measurements. The installation (and removal) of the hopper is performed by pumping out the sediment around it through an opening in its lowest point. As the removal of the material causes a rather significant disturbance in the bed, measurements performed soon after the hopper has been lowered into place are not representative of normal conditions. For this reason, all measurements given in this report were made after the hopper had been in place at least for 1 week.

The position of the water level for any one discharge is controlled by the conditions of the flow downstream from the measuring section. It is, therefore, important that the bed should not be significantly changed downstream from the hopper. For this reason the bed load taken out of the stream for measurement must be returned to the bed below the hopper after the measurement; otherwise, a certain amount of scour will occur downstream which in turn will increase the slope and transportation upstream from the hopper.

It may be mentioned that the depth of the sand bed above bedrock in Mountain Creek was too small to permit the whole width to be covered with one hopper, therefore, twin hoppers were installed, each 6 feet long. In operation they were pumped alternately for 1-minute periods, thus keeping both hoppers always practically empty.

The measuring tank

From the hopper, the sediment, together with a necessary amount of water, is drawn through the 3-inch suction pipe, then through a flexible 3-inch hose into a 3-inch centrifugal pump driven by a gasoline motor. From there it is pumped into the measuring or weighing tank. This tank, which is approximately 4 feet in diameter and of welded construction (figure 3), is always filled with water during the measuring operation. The sand-water mixture enters the tank at the center of the top, the sand settling to the bottom and the water overflowing along the entire rim into a wasteway which is rigidly attached to the frame support. The tank itself is supported from the frame by a strong steel spring. The elongation of this spring indicates the amount of water inside the tank that has been replaced by sediment. The elongation of the spring is amplified by a lever system and recorded on the chart of a water-stage recorder, modified for the purpose. A continuous record of the amount of sediment collected in the tank is therefore available and permits the determination of the rate of inflow of sediment for any time. When the tank is filled to capacity, the sediment is flushed back into the river through a 3-inch valve, located at the vertex of the conical bottom. The flushing of a full tank and its refilling with water takes less than 2 minutes. Its capacity is a little more than one cubic yard or 1500 pounds under water of sand with a specific gravity of 2.67. The lever system on the recorder has been adjusted so that the recorder on the chart can be read directly in pounds under water.

In order to keep the impact of the inflowing water and sand from affecting the recording mechanism, a special separator has been provided and attached directly to the frame support. This separator consists of a baffling system that stills the inflowing water.

The water leaves the tank overflowing along the entire rim and causes a rise of the water surface in the tank amounting to an overload of 15-20 pounds on the recording system. As the amount of overload is very constant with a reasonably constant discharge, the increase in weight does not affect rates of bed-load movement as determined from the recorder chart. In order to check on the constancy of the discharge, a flow measuring elbow (4) was built into the pressure line leading from the pump to the tank.

During the measurements in Mountain Creek, a pump and measuring tank were mounted permanently in that location. Afterwards it seemed preferable to mount the complete apparatus on a truck in order to have it strictly portable. This seemed especially important because pumping in several localities with the same equipment was desirable and also because some necessary calibration tests could not be performed very satisfactorily in some locations. But even before it was mounted permanently on a truck it would be considered movable as its transfer from one place to another could be performed by two men with a truck in 1 day. An additional hopper, of course, must be installed at each additional pumping location.

Calibration of the tank

The operation and efficiency of the weighing tank can be tested by two types of calibration test:

(1) Calibration of the weighing mechanism

(2) Calibration of the separator

The first calibration is most conveniently performed by starting with a tank filled to capacity with sand and water. The sand-water mixture then is gradually flushed out of the 3-inch valve into buckets and weighed. After the removal of each bucket, the recorder is read and this reading compared with the cumulative amount that was removed. Figure 4 shows a calibration of this type. The 45-degree line represents the points where the recorder shows the true values. The relatively small deviations from this line indicate the general reliability of the recording mechanism including spring, lever system and recorder. Repeated recalibration of the same type gave similar results.

The second calibration is intended to determine the efficiency of the separator by measuring the percentage of different grain sizes that are carried over the rim of the tank at various pump discharges. Each

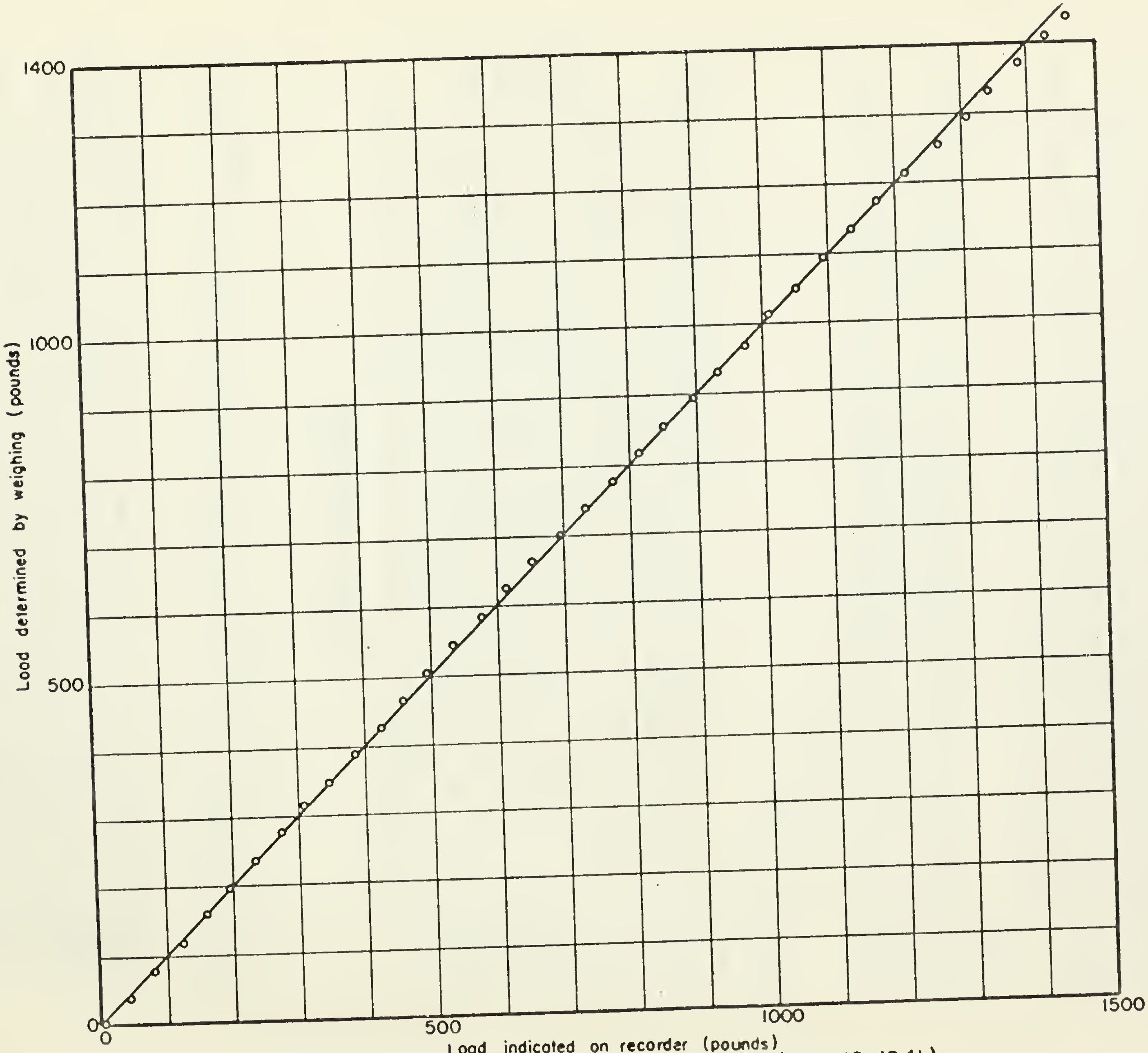

Figure 4-Calibration of the weighing mechanism (July 16, 1941.)

test was started by pumping water through the tank at a known rate. A measured amount of sand of a certain grain size then was fed into the suction hose without disturbing the flow. After 5 more minutes of continuous pumping of clear water, the pump was stopped and the amount of sand in the tank collected, measured, and compared with the original amount. The diagram in figure 5 gives the results of these calibrations and is used for correcting the mechanical analyses of the samples of trapped sediment.

A calibration of the flow-measuring elbow completes the test of the pumping unit. The results of these tests are shown in figure 5, calibration of the separator.

HYDRAULIC MEASUREMENTS

The measurements of bed-load transportation in Mountain Creek were made for the purpose of extending the knowledge of the law of transportation from flume studies to a natural flow of the size of a large flume. The laws of transportation are commonly expressed in two independent parts. The first part determines the rate of sediment transportation on a certain bed by a certain flow, usually called the transportation law. The second part refers to the change of the bed roughness caused by a certain rate of transportation, usually called the friction law. It must be borne in mind that the two parts of the problem are really independent and that both laws must be used in a successful attack on bed-load problems. These two laws constitute relationships between the rate of transportation and the flow and, therefore, measurements of the flow as well as of the transportation are needed for their application.

General problems

The transportation of sediment is generally expressed as a rate showing the amount of sediment moving through a certain cross section during a unit of time. Usually, little attention is given to the velocity of the particles or the thickness of the layer in movement. This study follows the same conception and therefore considers only the bulk movement of sediment into a section, that is, the section of the hopper.

In attempting to relate this measurement of bed-load transportation to the flow, it would seem most logical to describe both in the same section. Theoretically, this is possible as the local velocity distribution fully describes the flow. Practically, this description is very unsatisfactory for three reasons: (1) There are significant differences between the average velocities in different verticals of the section making it impossible to describe the distribution by any one vertical. (2) Because of changes in the movable bed, the velocity at any point will fluctuate irregularly, making the distribution of the

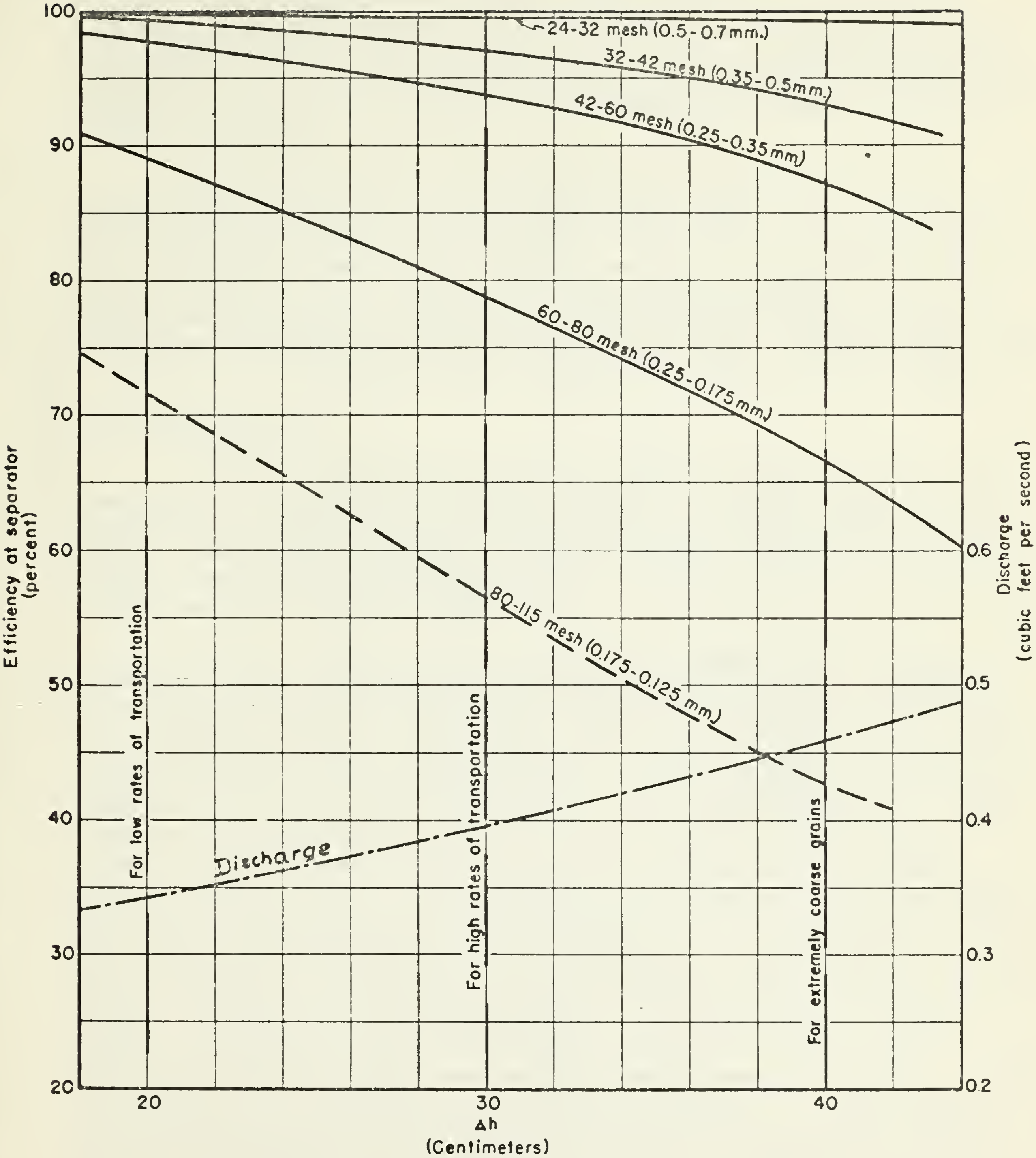

Figure 5 - Calibration of the separator tank. (Jan. 23, 1942)

local velocity extremely difficult, if not impossible, to determine. (3) Small changes in the velocity distribution are highly significant in the hydraulic sense, making extremely accurate measurements necessary. For these reasons, it has been found much more convenient to describe the flow in the manner of conventional hydraulics by the average velocity, slope, and depth.

To obtain the slope it was necessary to extend the hydraulic measurements over the whole reach (in Mountain Creek over a reach 750 feet long), and because of the natural irregularities in the stream, both the cross section and average velocity were taken as averages for the whole stretch. Localization of the sediment measurements in the lower end of the stretch was possible only because of the absence of any significant change in the position of the bed during the measuring period, thus indicating constant average transportation over the whole reach.

The cross section

A system of 16 cross sections was used to measure the slope and position of the bed and of the water surface. The first cross section, designated as range 0, was situated about 1 foot upstream from the hopper. Ranges 1-15 followed upstream at 50-foot intervals. Thus, the range number gives the distance of the cross sections upstream from the hopper measured in 50-foot intervals.

Each range was marked by two range ends consisting of a wooden stake driven into the bank and a wire hook attached to it. A movable wire chain with 1.5-foot links was connected with the wire hooks to mark the measuring points, 1.5 feet apart. This method permitted the exact relocation of the same measuring points each time, which is essential to making the different sets of measurements directly comparable. On each range all the points were recorded that were on the movable bed. The banks, which were rather steep and covered with vegetation, were not measured. The distance between the end points and the bank varied between 0 and 1.5 feet. In averaging the measurements of any one range, all points, therefore, were given equal weight. Besides the points on the bed, the measurements included a reading of the elevations of the water surface in each section as a check for the stage recorders and for the detection of accidental error. With two set-ups of a surveyor's level, elevations could be determined at all ranges in the 750-foot reach. An adjustable rod, which permitted elevations to be read directly, proved of great convenience in making these observations. Thus, all figures in the field book are directly comparable. A steel pipe, driven into the ground in a safe place off the stream, served as datum for all elevations and was given the assumed, but approximately true, elevation of 812.00 feet. Complete measurements on the ranges were made only at low stages because the bed is too easily disturbed when the sand is moving at higher rates to permit satisfactory measurements. Water-surface profiles were measured from the banks during higher stages and showed a uniform slope over the entire length of the reach (figure 6).

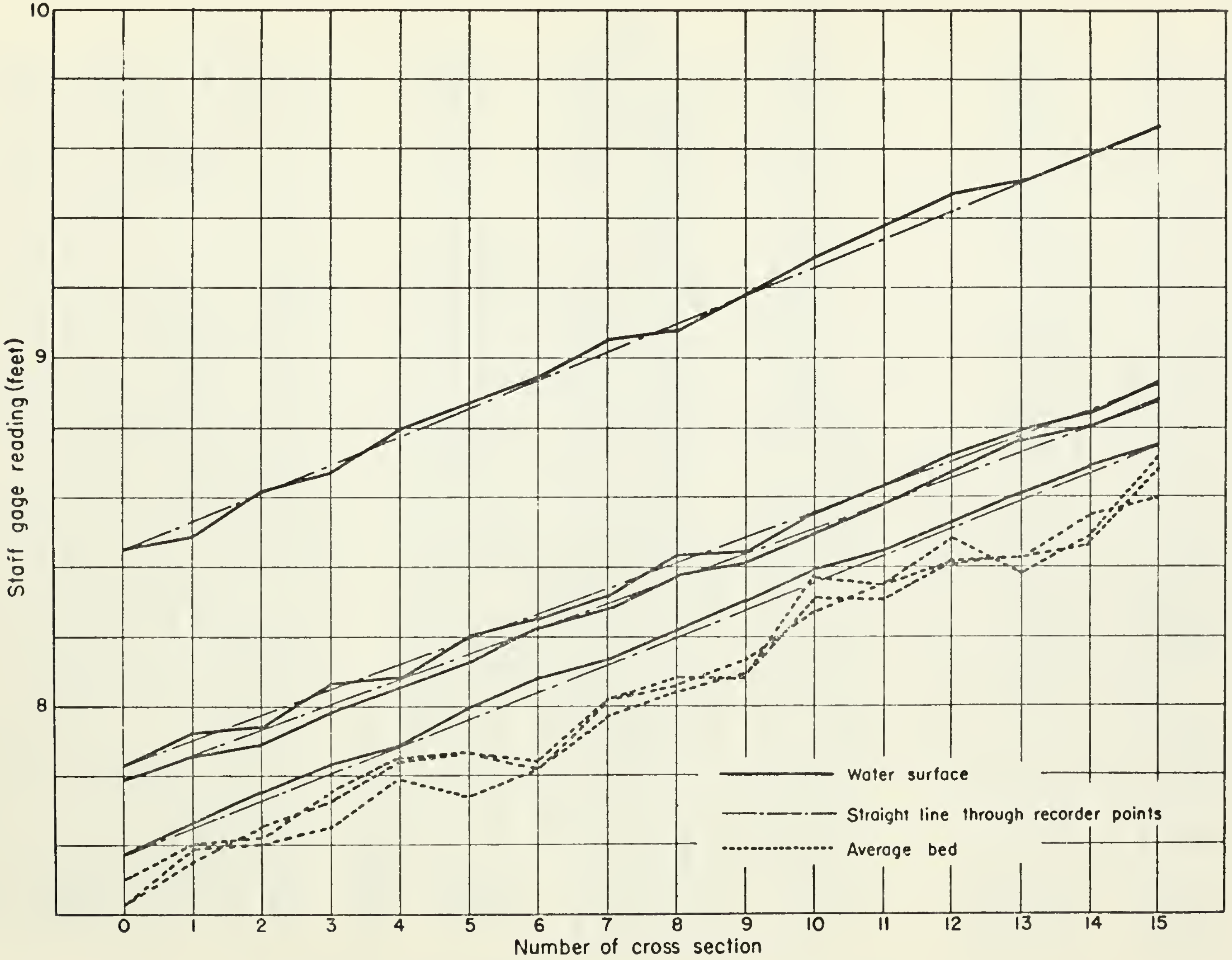

Figure 6 - Approximation of the water surface by a straight line

It was, therefore, concluded that a straight line through the range 0 to 15, as given by the two stage recorders located at those two ranges, would satisfactorily describe the water surface. Only for times of fast changing discharge does this method seem not to be satisfactory. Table 1 shows an example of a set of cross sections. The line "area" gives the calculated cross-sectional areas, assuming vertical banks. This value is very close to correct as the water depth during the measurements was small. The values in this line are calculated as the sum of the water depths at the different points of the range multiplied by the distance of 1.50 feet, and therefore, do not include the points of the bed above water surface. In the calculation of the "average bed," however, these points are included. The slope of the bed was calculated by the method of least squares.

The water surface

It has been mentioned previously that the system of range measurements was used to obtain regularly-spaced measurements of the water surface and that the water surface could be considered as a straight line within the measuring reach for the higher stages.

As the stage in a natural river is changing almost constantly, it was necessary to record the water surface at least at one point continuously. Especially satisfactory results were obtained in this case where two automatic stage recorders were installed, one at range 0 and the other at range 15. A comparison of the records from the two recorders gave a good check on the stage and a continuous record of the slope of the water surface.

Bed-load formulas and friction laws usually contain the slope of the energy grade line as one of the hydraulic variables. In a flow that approaches normal flow, the slope, S_e, of the energy grade line can be very closely approximated by the formula,

$$S_e = S_w + \alpha \frac{d}{dx} \left(\frac{V^2}{2g}\right)$$

where S_w is the slope of the water surface, V the average velocity in a cross section, x the horizontal distance which increases upstream, and α the factor which corrects for the increase in kinetic energy due to the variation of velocity within the cross section.

S_w is obtained by dividing the difference in the water-surface elevations, N_w, between range 0 and range 15 by 750 feet, the distance between the ranges. In Mountain Creek N_w was usually approximately 1 foot. The recorder charts could be read easily to 0.01 ft. thus permitting the slope to be determined with an average error within about 1%.

Table 1.--Cross section measurements taken on Mountain Creek August 29, 1941

Range	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	Average
Time	12:06	12:09	12:13	12:18	12:21	12:27	12:31	12:34	12:45	12:48	12:51	12:56	13:00	13:04	13:06	13:10	
W. S.	7.76	7.85	7.88	7.97	8.04	8.12	8.23	8.285	8.37	8.41	8.48	8.565	8.64	8.74	8.82	8.90	8.316
Pt. 1																	
2	7.62				7.70			8.07			8.09		8.27				
3	7.59	7.64	7.24		7.63	7.79		7.99	8.04	8.44*	7.93		8.36	8.48	8.55		
4	7.54	7.62	7.35	8.18*	7.63	7.80	7.79	7.91	7.91	8.42*	7.80	8.57*	8.37	8.40	8.41	8.75	
5	7.57	7.66	7.59	8.01*	7.66	7.85	7.79	7.93	7.96	8.31	7.89	8.69*	8.38	8.33	8.42	8.66	
6	7.51	7.64	7.66	7.83	7.69	8.01	8.00	7.95	8.01	8.04	8.39	8.60*	8.35	8.23	8.38	8.64	
7	7.50	7.56	7.66	7.80	7.76	8.09	8.01	8.00	8.04	8.03	8.40	8.52	8.34	8.33	8.39	8.62	
8	7.47	7.56	7.74	7.30	7.86	7.81	7.93	8.07	8.13	7.95	8.50*	8.39	8.36	8.48	8.50	8.67	
9	7.41	7.58	7.84	7.79	8.09*	7.77	7.79	8.21	8.19	7.92	8.46	8.32	8.52	8.53	8.61	8.65	
10	7.38	7.58	7.95*	7.71	8.15*	7.78	7.80	8.25	8.19	7.89	8.49*	8.22	8.60	8.44	8.79	8.60	
11	7.38	7.60		7.64		7.79	7.75	8.25	8.11	8.01	8.69*	8.03	8.54		9.00*	8.49	
12		7.64		7.34				8.00				7.96	8.38			8.38	
13		7.66		7.18								8.08					
Area	3.945	3.915	3.12	4.005	3.525	3.585	4.47	3.757	4.125	4.080	3.60	3.652	3.855	4.050	3.765	3.96	3.838
Av.Bed	7.50	7.61	7.63	7.73	7.80	7.85	7.86	8.06	8.09	8.11	8.26	8.34	8.41	8.40	8.56	8.61	8.051

*Above Water Surface

Slope; W.S. 0.001520
Bed 0.001476

The average value for $V^2/2_g$ in Mountain Creek varied with the stage during the measuring period from .02 foot to .09 foot. Even a change of this value of 10% from one end of the reach to the other would result in less than 1% error for S_e; therefore, the velocity head correction was neglected in the various calculations. Thus, for practical purposes, the energy grade line may be considered parallel to the water surface.

The representative cross section of the bed

The slope of the energy grade line as determined above represents an average value for the whole measuring reach. The other hydraulic variables such as the average velocity and the dimensions of the cross section also have to be determined as averages over the same reach. The average or representative cross section was obtained from 3 selected sets of cross-section measurements in the following manner and used in connection with all discharges.

The slopes of the 3 beds were first averaged, this average being called the representative slope, S_r, of the bed. Then a number of parallel planes were established through certain levels in the middle of the reach, planes that were horizontal across the creek and sloping with S_r in the direction of the flow. Each one of these planes represented an ideal water surface, which permitted the area and the wetted perimeter to be determined in each one of the 48 cross sections. Figure 7 illustrates this procedure and shows a typical cross section with the two range ends and the hooks which support the marker chain. The line A-D is the above mentioned water surface, projected with the slope S_r from the center of the reach. The actual cross section has the area A-B-C-D, the wetted perimeter of the bed B-C, and the wetted perimeter of the banks A-B and C-D. This actual cross section could be determined only by a large number of measuring points plotted on a scale drawing. As any one cross section is not representative for a considerable length of the river due to normal irregularities in a natural river, it seemed necessary to use all 16 of the ranges. This relatively large number, however, suggested the application of a simplified method of computation. Thus, the actual area A-B-C-D is approximated by the shaded area in figure 7. This approximation is obtained by extending the level of each measuring point on the bed 0.75 foot on each side of the measuring point, and by the addition of a 45° triangular area on each end of this series of rectangles. This substitute area is rather complicated to define, but permits a simple calculation of all the significant variables. If d is the water depth in feet at any one of the n measuring points in the section, the width of the bed is 1.5 n feet. The cross-sectional area is:

$$A = 1.5 \sum_{n} d + 1/2\ (d_e^2 + d_{e_1}{}^2)$$

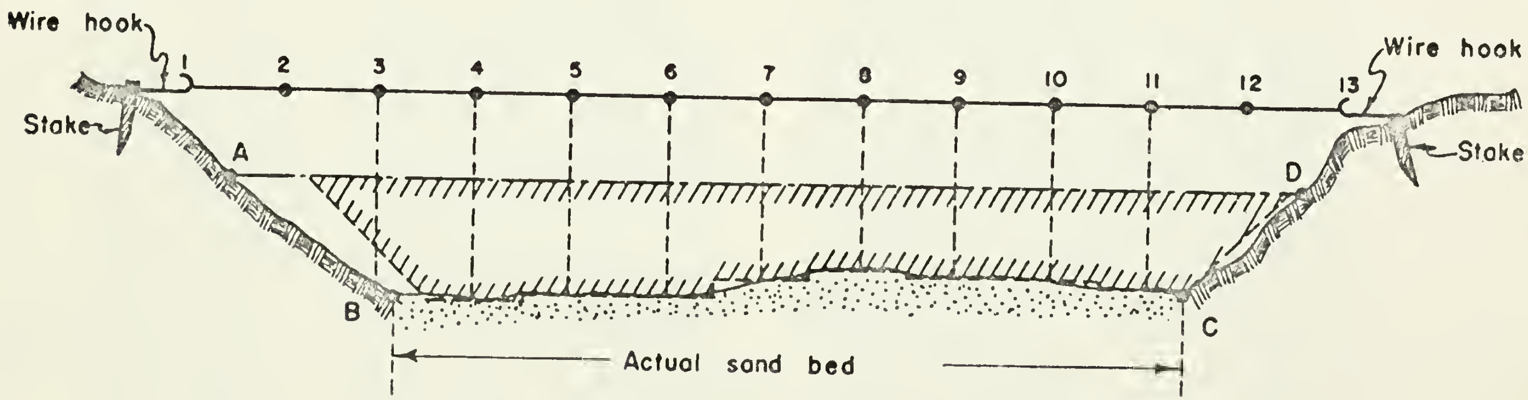

Figure 7— Method of cross sectioning used to determine the representative cross section of the bed.

where d_e and d_{el} are the depths at the two end measuring points, i.e., points 4 and 11 in figure 7. The perimeters of the banks, therefore, are:

$$\sqrt{2}\, d_e \quad \text{and} \quad \sqrt{2}\, d_{el}$$

In figure 7 the last measuring point falls very close to 0.75 ft inside of point C, thus causing the approximation to check rather closely with the true section. On the left side, point 3 falls just outside the bed, causing the approximation to be too short. If point 3 were just inside the bed, the approximation would be the same amount too long. It is assumed, therefore, that over the 16 ranges these deviations will compensate one another for all practical purposes.

For each stage the areas and the wetted perimeters of the banks and of the bed have been averaged for the 48 cross sections, and these averages plotted in figure 8 against the stage as measured at the middle of the measuring reach. This diagram, therefore, constitutes the description of the representative cross section of the measuring reach and is used in all calculations. Even for flows where the slope of the water surface differs from S_w, figure 8 can be used because the representative stage is measured in the middle of the measuring reach which is midway between the two stage recorders which give a measure of slope and stage at any one instance. It may be mentioned that the representative cross section itself probably cannot be plotted from these curves, but this does not prove them to be incorrect.

The velocity measurement

In selecting a method of measuring the velocity of the water, two conditions must be kept in mind: (1) the average velocity over the whole measuring reach is to be determined, and (2) the stage, and with it the velocity is changing continuously. The most desirable method for this measurement would be to install a weir equipped with an automatic recorder. Unfortunately, however, local conditions and the cost of the installation made this method impracticable. Instead, the surface velocity was measured over the whole measuring reach by means of floats at rather frequent intervals at various stages, and the average velocities calculated by a reduction factor of 0.85. An occasional determination of the discharge with a Pitot Tube showed very close agreement with the float measurements. The results of the various velocity measurements are plotted in figure 8 against the stage at the center of the measuring reach. A curve drawn through the plotted points, rather than the individual measurements, was used in all calculations. The use of the averaged curve appeared justified as the deviation of the points from the curve seemed purely accidental.

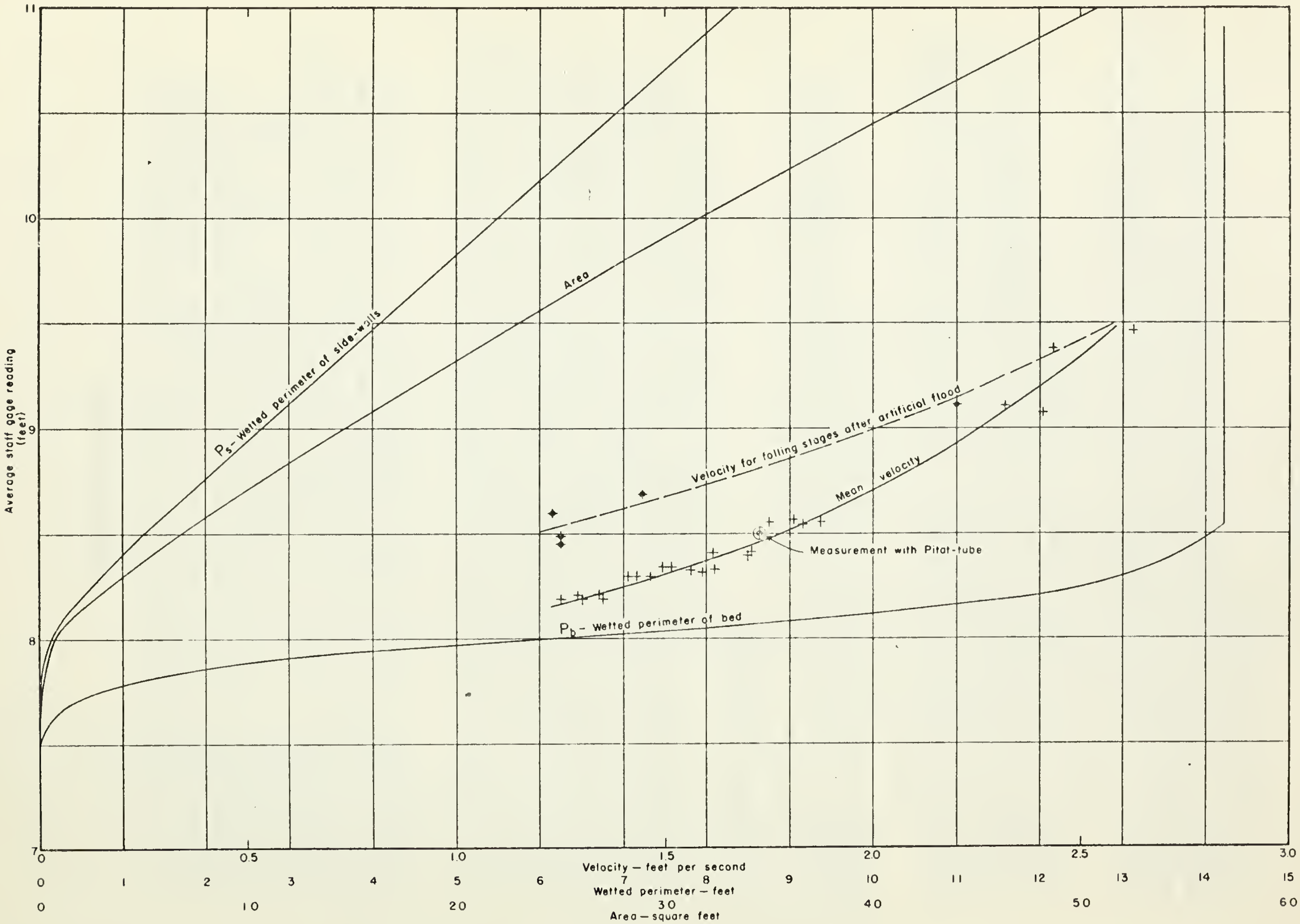

Figure 8 - Curves of area, wetted perimeter, and mean velocity for an average section of Mountain Creek.

EXPERIMENTAL DATA

The measurements made between August 19 and November 1, 1941, covered a rather long period of dry weather. The total rainfall during the period was only 3.55 inches and of this amount, 1.55 inches were concentrated in one storm on October 31. The rather warm temperature and the lack of high floods were conducive to the establishment of a luxurious growth of various plants on the stream banks. This quick growth probably explains the relatively high roughness of the banks as indicated in the analysis of the data. Besides the various observations from which figure 8 was prepared the observational data included measurements of the rate of bed-load transportation and records from the two stage recorders. Table 3 gives a complete summary of all observed and calculated data from the Mountain Creek study.

During the period when observations were being made, two floods of about an hour's duration each were artifically produced by releasing water from the small mill pond upstream from the measuring reach. The falling stages during these artificial floods were observed to decrease much more rapidly than during natural floods. Because of this relatively rapid decrease in stage, the flow was unable to affect the same readjustment in the configuration of the bed surface as under natural conditions; consequently the bed configuration at a certain stage was representative of a configuration characteristic of a much higher stage during a natural flood. Measurements made during these unnatural conditions are considered separately; as, for instance, the velocity curve shown in figure 8.

The sediment measurements

In table 3, column 1 gives the date of the observation; column 2 the time of the measurement; column 3 the duration in minutes of the particular measuring period; and column 5 the rates of transportation as determined from the records of the weighing tank.

Although the weighing tank gives a continuous record, the pumping periods were divided into measuring periods ranging from 10 to 60 minutes for the convenience in calculation. The short periods were used for high stages, when stage and rate of transportation were changing rapidly, and the longer periods were used for low stage, when rates were small and a reasonably exact reading of the accumulation of sediment in the tank was possible only with longer periods of collection.

Although separate mechanical analyses for determining the composition of the transported material were not made for each measuring period, a sufficient number of samples were collected and analyzed to give a reasonably accurate measure of the composition of the bed load. Each time the tank was flushed, a sample of the sediment was taken for a mechanical analysis, so that each analysis represents the composite material from several measuring periods.

TABLE 3

SUMMARY OF DATA ON BED-LOAD TRANSPORTATION AND BED ROUGHNESS OBSERVED IN MOUNTAIN CREEK, GREENVILLE, S. C.

Date (1)	Hour (2)	Duration of run, minutes (3)	Water temperature °C (4)	Transportation, Q_s' pounds (dry, under water) per hour (5)	Stage at range 15 * feet (6)	Stage at range 0 feet (7)	Average stage, feet (8)	Slope, S (9)	Mean velocity, V feet per second (10)	Area, A - square feet (11)	Wetted perimeter of banks, P_w - feet (12)	$n_w^{3/2}$ for $n_b = 0$ (13)	Wetted perimeter of bed, P_b - feet (14)	$n_b^{3/2}$ for $n_w = 0$ (15)	$\phi = 0.00425 \frac{Q_s'}{P_b}$ (16)	n_w=0.068 R_w (17)	R_b (18)	ψ (19)	n_b (20)	n_w=0.062 R_w (21)	R_b (22)	ψ (23)	n_b (24)	n_w=0.056 R_w (25)	R_b (26)	ψ (27)	n_b (28)	n_w=0.050 R_w (29)	R_b (30)	ψ (31)	n_b (32)	n_w=0.044 R_w (33)	R_b (34)	ψ (35)	n_b (36)
8-19-41	11:15	60	25	60	8.80	7.77	8.28	.00137	1.46	3.8	0.73	.0301	12.87	.00216	.0198	2.43	0.157	16.6	.0110	2.11	0.176	14.8	.0118	1.82	0.192	13.6	.0126	1.53	0.208	12.5	.0132	1.26	0.224	11.6	.0139
"	12:15	60		60	8.81	7.77	8.29	.00139	1.48	4.0	0.74	.0392	12.95	.00224	.0197	2.46	0.168	15.3	.0114	2.15	0.186	13.8	.0122	1.83	0.204	12.6	.0130	1.55	0.220	11.6	.0136	1.28	0.236	10.9	.0143
"	13:15	60		60	8.81	7.78	8.29	.00137	1.48	4.0	0.74	.0387	12.95	.00221	.0197	2.48	0.167	15.6	.0113	2.16	0.185	14.1	.0120	1.85	0.203	12.8	.0128	1.56	0.220	11.8	.0135	1.29	0.235	11.1	.0142
"	14:15	60		70	8.82	7.78	8.30	.00139	1.50	4.1	0.76	.0383	13.04	.00223	.0228	2.51	0.168	15.3	.0113	2.19	0.187	13.7	.0120	1.87	0.205	12.5	.0128	1.58	0.222	11.6	.0135	1.31	0.238	10.8	.0142
"	15:15	60		60	8.81	7.76	8.28	.00140	1.46	3.8	0.72	.0393	12.87	.00220	.0198	2.39	0.162	15.7	.0113	2.08	0.179	14.2	.0121	1.78	0.196	13.0	.0129	1.50	0.211	12.1	.0135	1.24	0.226	11.3	.0142
8-25-41	10:15	30	24	260	9.03	7.90	8.47	.00150	1.74	6.4	1.20	.0321	13.96	.00276	.0792	2.96	0.204	11.7	.0114	2.58	0.237	10.0	.0124	2.20	0.269	8.85	.0138	1.87	0.296	7.99	.0148	1.54	0.326	7.30	.0157
"	10:45	30		160	9.02	7.90	8.46	.00149	1.73	6.3	1.17	.0324	13.94	.00272	.0488	2.94	0.205	11.7	.0116	2.55	0.238	10.1	.0128	2.20	0.262	8.97	.0138	1.85	0.297	8.07	.0140	1.53	0.324	7.39	.0157
"	11:15	30		180	9.00	7.86	8.43	.00151	1.69	5.8	1.10	.0332	13.80	.00265	.0554	2.81	0.196	12.1	.0115	2.45	0.225	10.5	.0127	2.10	0.253	9.34	.0137	1.78	0.278	8.50	.0146	1.47	0.303	7.80	.0154
"	11:45	30		140	8.96	7.85	8.41	.00151	1.66	5.6	1.04	.0349	13.70	.00265	.0434	2.73	0.202	11.7	.0120	2.37	0.229	10.3	.0129	2.04	0.251	9.31	.0138	1.72	0.278	8.50	.0147	1.42	0.302	7.85	.0155
"	12:15	30		180	8.96	7.84	8.40	.00149	1.65	5.5	1.02	.0348	13.65	.00260	.0560	2.74	0.198	12.1	.0118	2.39	0.225	10.7	.0129	2.04	0.250	9.58	.0138	1.72	0.274	8.74	.0146	1.42	0.297	8.07	.0155
"	12:45	30		160	8.95	7.82	8.39	.00150	1.63	5.3	0.99	.0356	13.60	.00259	.0500	2.67	0.195	12.2	.0118	2.33	0.220	10.8	.0129	2.00	0.240	9.92	.0136	1.69	0.267	8.91	.0147	1.39	0.289	8.24	.0154
"	13:15	30		120	8.94	7.82	8.38	.00149	1.62	5.2	0.97	.0356	13.55	.00255	.0376	2.66	0.193	12.4	.0118	2.31	0.218	11.0	.0128	1.99	0.241	9.94	.0137	1.68	0.263	9.11	.0145	1.38	0.285	8.41	.0153
"	13:45	30		100	8.93	7.81	8.37	.00149	1.60	5.0	0.94	.0360	13.50	.00251	.0315	2.62	0.188	12.7	.0118	2.28	0.212	11.3	.0128	1.95	0.235	10.2	.0136	1.65	0.255	9.40	.0144	1.36	0.276	8.68	.0152
"	14:15	30		120	8.92	7.81	8.36	.00148	1.59	4.9	0.92	.0368	13.45	.00252	.0379	2.60	0.186	13.0	.0117	2.26	0.210	11.5	.0127	1.94	0.232	10.4	.0136	1.64	0.252	9.57	.0144	1.35	0.272	8.87	.0151
"	14:45	30		120	8.91	7.80	8.35	.00148	1.57	4.7	0.89	.0367	13.39	.00244	.0381	2.55	0.182	13.3	.0117	2.21	0.204	11.8	.0126	1.91	0.224	10.8	.0134	1.61	0.244	9.89	.0142	1.33	0.263	9.17	.0150
"	15:15	30		120	8.90	7.79	8.34	.00148	1.56	4.6	0.86	.0375	13.33	.00242	.0383	2.53	0.182	13.3	.0117	2.20	0.203	11.9	.0126	1.89	0.223	10.8	.0135	1.59	0.243	9.93	.0143	1.32	0.260	9.26	.0149
8-29-41	9:30	25	21.5	75	8.88	7.77	8.32	.00148	1.53	4.4	0.82	.0388	13.19	.00241	.0242	2.46	0.181	13.3	.0119	2.15	0.200	12.1	.0127	1.84	0.219	11.0	.0135	1.56	0.237	10.2	.0143	1.28	0.254	9.50	.0149
"	10:15	60		80	8.89	7.77	8.33	.00149	1.54	4.5	0.84	.0385	13.25	.00244	.0257	2.46	0.184	13.0	.0121	2.15	0.203	11.8	.0128	1.85	0.222	10.8	.0137	1.56	0.241	9.94	.0145	1.28	0.258	9.35	.0151
"	11:15	60		100	8.89	7.76	8.32	.00151	1.53	4.4	0.82	.0394	13.19	.00215	.0322	2.42	0.183	12.9	.0122	2.11	0.202	11.7	.0130	1.81	0.221	10.7	.0138	1.53	0.235	9.93	.0145	1.26	0.255	9.27	.0152
"	12:15	60		100	8.90	7.76	8.33	.00152	1.54	4.5	0.84	.0393	13.25	.00249	.0321	2.43	0.185	12.7	.0122	2.12	0.205	11.5	.0131	1.82	0.224	10.5	.0139	1.53	0.243	9.67	.0147	1.26	0.260	9.03	.0153
"	13:15	60		125	8.91	7.75	8.33	.00155	1.54	4.5	0.84	.0396	13.25	.00257	.0401	2.40	0.187	12.3	.0124	2.09	0.207	11.1	.0133	1.79	0.226	10.2	.0141	1.51	0.244	9.44	.0148	1.25	0.260	8.86	.0155
"	14:15	60		90	8.92	7.75	8.33	.00156	1.54	4.5	0.84	.0398	13.25	.00252	.0289	2.38	0.189	12.1	.0126	2.07	0.205	11.0	.0134	1.78	0.227	10.1	.0142	1.50	0.245	9.34	.0150	1.24	0.261	8.77	.0156
"	15:15	60		110	8.92	7.74	8.33	.00157	1.54	4.5	0.84	.0399	13.25	.00253	.0353	2.37	0.189	12.0	.0126	2.06	0.209	10.9	.0135	1.77	0.227	10.0	.0142	1.50	0.245	9.28	.0150	1.24	0.261	8.71	.0156
9-3-41	9:22	60	25	35	8.76	7.62	8.19	.00152	1.30	2.7	0.50	.0508	11.65	.00178	.0128	1.90	0.150	15.7	.0126	1.65	0.161	13.6	.0132	1.41	0.171	13.7	.0137	1.20	0.180	13.0	.0142	0.93	0.185	12.4	.0147
"	9:57	10		1080	9.06	7.90	8.48	.00155	1.76	6.6	1.23	.0317	14.00	.00278	.328	2.93	0.214	10.8	.0119	2.55	0.247	9.32	.0131	2.19	0.279	8.26	.0142	1.85	0.309	7.45	.0152	1.52	0.338	6.81	.0162
"	10:07	10		2880	9.78	8.44	9.11	.00178	2.34	16.5	3.00	.0242	14.22	.00511	.861	4.04	0.309	6.43	.0123	3.52	0.418	4.77	.0150	3.01	0.525	3.80	.0175	2.56	0.620	3.22	.0196	2.10	0.717	2.78	.0215
"	10:27	10		3360	10.09	8.70	9.39	.00185	2.53	21.2	3.79	.0224	14.22	.00595	1.004	4.44	0.307	6.29	.0115	3.86	0.462	4.18	.0151	3.30	0.611	3.16	.0182	2.80	0.745	2.59	.0208	2.30	0.878	2.20	.0231
"	10:37	10		3120	10.16	8.75	9.45	.00188	2.56	22.1	3.95	.0224	14.22	.00622	.932	4.45	0.325	5.85	.0119	3.87	0.486	3.91	.0156	3.32	0.639	2.97	.0187	2.81	0.781	2.43	.0215	2.31	0.920	2.06	.0239
"	10:47	10		2040	10.16	8.79	9.47	.00183	2.58	22.5	4.01	.0217	14.22	.00611	.610	4.59	0.289	6.75	.0108	3.99	0.457	4.27	.0146	3.42	0.618	3.16	.0178	2.69	0.767	2.54	.0206	2.39	0.908	2.15	.0231
"	10:57	10		1530	10.18	8.74	9.46	.00192	2.57	22.4	3.99	.0226	14.22	.00634	.457	4.39	0.323	5.42	.0124	3.81	0.506	3.67	.0161	3.29	0.652	2.85	.0191	2.76	0.801	2.32	.0213	2.28	0.933	1.99	.0242
"	11:07	10		1530	10.06	8.66	9.36	.00187	2.51	20.7	3.70	.0229	14.22	.00595	.457	4.33	0.329	5.80	.0122	3.76	0.477	4.00	.0156	3.23	0.615	3.10	.0185	2.74	0.743	2.57	.0210	2.25	0.870	2.15	.0234
"	11:17	10		720	9.87	8.49	9.18	.00184	2.24	17.7	3.20	.0266	14.22	.00597	.215	3.70	0.412	4.70	.0157	3.24	0.516	3.75	.0183	2.77	0.621	3.12	.0207	2.34	0.718	2.70	.0227	1.92	0.813	2.38	.0248
"	11:27	10		240	9.57	8.29	8.93	.00171	1.90	13.5	2.49	.0315	14.22	.00551	.0717	3.05	0.415	5.04	.0180	2.65	0.485	4.30	.0200	2.28	0.550	3.80	.0217	1.92	0.613	3.41	.0234	1.58	0.673	3.10	.0249
"	11:37	10		60	9.33	8.09	8.71	.00165	1.56	10.0	1.90	.0400	14.22	.00535	.0184	2.33	0.392	5.51	.0208	2.03	0.432	5.00	.0221	1.74	0.471	4.59	.0239	1.47	0.507	4.26	.0246	1.21	0.542	3.99	.0258
"	11:47	10		60	9.19	7.97	8.58	.00163	1.32	8.0	1.50	.0517	14.22	.00545	.0179	1.83	0.370	5.91	.0235	1.60	0.394	5.55	.0245	1.37	0.418	5.24	.0255	1.15	0.441	4.96	.0264	0.95	0.462	4.74	.0272
"	11:57	10		30	9.11	7.89	8.50	.00163	1.20	6.9	1.29	.0545	14.09	.00500	.0090	1.58	0.345	6.35	.0240	1.38	0.363	6.04	.0255	1.18	0.382	5.74	.0264	1.00	0.398	5.50	.0270	0.82	0.415	5.28	.0279
9-4-41	12:15	30	25.5	180	8.92	7.72	8.32	.00160	1.53	4.3	0.82	.0401	13.19	.00250	.0580	2.31	0.182	12.3	.0124	2.02	0.199	11.2	.0131	1.73	0.216	10.3	.0140	1.46	0.231	9.55	.0146	1.21	0.251	8.93	.0152
"	12:45	30		160	8.91	7.72	8.32	.00159	1.53	4.3	0.82	.0400	13.19	.00249	.0516	2.33	0.181	12.4	.0124	2.03	0.200	11.2	.0132	1.74	0.218	10.3	.0140	1.47	0.235	9.55	.0146	1.21	0.251	8.94	.0153
"	13:15	30		180	8.92	7.73	8.32	.00159	1.53	4.3	0.82	.0400	13.19	.00249	.0580	2.33	0.181	12.4	.0124	2.03	0.200	11.2	.0132	1.74	0.218	10.3	.0140	1.47	0.235	9.55	.0146	1.21	0.251	8.94	.0153
"	13:45	30		160	8.92	7.73	8.32	.00159	1.53	4.3	0.82	.0400	13.19	.00249	.0516	2.33	0.181	12.4	.0124	2.03	0.200	11.2	.0132	1.74	0.218	10.3	.0140	1.47	0.235	9.55	.0146	1.21	0.251	8.94	.0153
"	14:15	30		200	8.92	7.74	8.33	.00158	1.55	4.4	0.84	.0390	13.25	.00247	.0642	2.38	0.181	12.5	.0122	2.07	0.201	11.2	.0131	1.78	0.219	10.3	.0139	1.50	0.237	9.54	.0147	1.23	0.252	8.93	.0153
"	14:45	30		160	8.92	7.74	8.33	.00156	1.55	4.4	0.84	.0390	13.25	.00247	.0526	2.38	0.181	12.5	.0122	2.07	0.201	11.2	.0131	1.78	0.219	10.3	.0139	1.50	0.237	9.54	.0147	1.23	0.252	8.93	.0153
"	15:15	30		220	8.98	7.81	8.39	.00156	1.64	5.3	0.99	.0362	13.60	.00264	.0688	2.62	0.199	11.5	.0122	2.29	0.223	10.3	.0132	1.96	0.241	9.26	.0141	1.66	0.263	8.50	.0149	1.36	0.292	7.66	.0155
"	15:45	30		240	9.14	7.91	8.52	.00163	1.80	7.1	1.34	.0322	14.15	.00305	.0721	2.92	0.225	9.73	.0123	2.55	0.263	8.42	.0136	2.19	0.294	7.45	.0147	1.84	0.328	6.68	.0158	1.52	0.358	6.11	.0168
"	16:15	30		400	9.22	8.02	8.62	.00161	1.91	8.6	1.61	.0295	14.22	.00334	.1195	3.21	0.241	9.20	.0121	2.80	0.288	7.70	.0136	2.40	0.333	6.65	.0150	2.03	0.375	5.91	.0163	1.67	0.416	5.32	.0174
"	16:45	30		400	9.23	8.01	8.62	.00163	1.91	8.6	1.61	.0297	14.22	.00336	.1195	3.19	0.244	8.97	.0122	2.78	0.290	7.55	.0138	2.39	0.334	6.55	.0151	2.01	0.377	5.80	.0164	1.66	0.417	5.25	.0175
"	17:15	30		560	9.21	8.01	8.61	.00161	1.90	8.5	1.58	.0299	14.22	.00332	.1674	3.19	0.243	9.12	.0122	2.78	0.289	7.67	.0137	2.39	0.332	6.68	.0151	2.01	0.374	5.93	.0163	1.66	0.413	5.37	.0174
"	17:45	30		500	9.19	8.00	8.60	.00159	1.89	8.4	1.55	.0301	14.22	.00328	.1494	3.20	0.242	9.29	.0122	2.79	0.287	7.82	.0136	2.39	0.330	6.80	.0149	2.01	0.372	6.05	.0162	1.66	0.410	5.48	.0173
"	18:15	30		340	9.16	7.99	8.58	.00157	1.87	8.0	1.50	.0298	14.22	.00314	.1016	3.18	0.227	10.0	.0118	2.77	0.270	8.41	.0132	2.37	0.313	7.26	.0145	2.00	0.352	6.46	.0157	1.65	0.385	5.85	.0166
"	18:45	30		380	9.15	7.97	8.56	.00158	1.85	7.8	1.45	.0307	14.22	.00314	.1136	3.11	0.231	9.77	.0120	2.71	0.272	8.30	.0134	2.33	0.311	7.26	.0147	1.96	0.349	6.47	.0158	1.62	0.383	5.90	.0168
"	19:15	30		380	9.15	7.97	8.56	.00158	1.85	7.8	1.45	.0307	14.22	.00314	.1136	3.11	0.231	9.77	.0120	2.71	0.272	8.30	.0134	2.33	0.311	7.26	.0147	1.96	0.349	6.47	.0158	1.62	0.383	5.90	.0168
"	19:45	30		320	9.11	7.94	8.52	.00156	1.80	7.2	1.34	.0316	14.15	.00300	.0961	3.02	0.223	10.3	.0120	2.64	0.259	8.84	.0133	2.26	0.295	7.76	.0145	1.91	0.326	6.98	.0156	1.57	0.360	6.36	.0166
"	20:15	30		300	9.10	7.93	8.52	.00156	1.80	7.2	1.34	.0316	14.15	.00300	.0901	3.02	0.223	10.3	.0120	2.64	0.259	8.84	.0133	2.26	0.295	7.76	.0145	1.91	0.328	6.98	.0156	1.57	0.360	6.36	.0166
"	20:45	30		340	9.09	7.90	8.50	.00159	1.78	6.9	1.29	.0325	14.09	.00298	.1026	2.92	0.222	10.4	.0122	2.55	0.256	8.78	.0134	2.19	0.289	7.78	.0146	1.84	0.321	7.00	.0156	1.52	0.351	6.40	.0166
"	21:15	30		360	9.07	7.90	8.48	.00157	1.76	6.6	1.24	.0325	14.00	.00285	.1093	2.90	0.215	10.6	.0120	2.54	0.246	9.25	.0132	2.17	0.279	8.15	.0143	1.83	0.309	7.36	.0152	1.51	0.338	6.73	.0162
"	21:45	30		240	9.06	7.89	8.47	.00157	1.74	6.4	1.20	.0332	13.96	.00285	.0731	2.85	0.213	10.7	.0120	2.49	0.244	9.32	.0132	2.14	0.274	8.30	.0143	1.80	0.306	7.49	.0153	1.48	0.331	6.86	.0162
9-16-41	11:35	130	21.5	24	8.79	7.68	8.23	.00158	1.38	3.1	0.60	.0457	12.35	.00222	.0083	2.00	0.154	14.7	.0123	1.74	0.166	13.6	.0130	1.50	0.178	12.7	.0136	1.26	0.190	11.9	.0142	1.04	0.200	11.3	.0147
9-18-41	10:50	160	21.5	32	8.77	7.59	8.18	.00161	1.28	2.6	0.48	.0544	11.50	.00227	.0118	1.76	0.153	14.5	.0133	1.53	0.162	13.7	.0138	1.32	0.171	13.0	.0143	1.11	0.180	12.3	.0148	0.91	0.158	11.8	.0153
10-22-41	10:50	60	20	20	8.72	7.59	8.15	.00151	1.23	2.2	0.41	.0545	10.80	.00207	.0079	1.74	0.138	17.1	.0126	1.51	0.146	16.2	.0130	1.30	0.154	15.3	.0135	1.10	0.162	14.6	.0140	0.90	0.170	13.9	.0144
"	11:25	10		450	9.00	7.98	8.49	.00136	1.88	6.7	1.25	.0267	14.05	.00237	.1361	3.56	0.160	16.4	.0086	3.11	0.200	13.1	.0100	2.66	0.240	10.9	.0113	2.25	0.277	9.46	.0124	1.86	0.311	8.44	.0134
"	11:35	10		1200	9.45	8.25	8.85	.00160	2.14	12.3	2.26	.0252	14.22	.00401	.3586	3.82	0.258	8.65	.0113	3.34	0.334	6.68	.0134	2.85	0.412	5.41	.0154	2.41	0.482	4.62	.0171	1.99	0.549	4.06	.0186
"	11:45	10		1300	9.66	8.35	9.00	.00175	2.26	14.7	2.68	.0250	14.22	.00470	.4124	3.89	0.301	6.78	.0124	3.40	0.393	5.19	.0148	2.91	0.485	4.20	.0170	2.45	0.572	3.56	.0190	2.56	0.646	3.16	.0205
"	11:55	10		1830	9.74	8.40	9.07	.00179	2.31	15.8	2.88	.0246	14.22	.00498	.5469	3.95	0.311	6.41	.0125	3.45	0.412	4.84	.0151	2.95	0.514	3.88	.0175	2.40	0.607	3.29	.0195	2.05	0.696	2.87	.0214
"	12:05	10		1290	9.77	8.44	9.10	.00177	2.33	16.4	2.97	.0243	14.22	.00507	.3855	4.04	0.310	6.51	.0123	3.51	0.420	4.80	.0150	3.02	0.523	3.86	.0174	2.55	0.621	3.25	.0195	2.10	0.715	2.82	.0214
"	12:15	10		1200	9.79	8.43	9.11	.00181	2.34	16.5	3.00	.0245	14.22	.00515	.3586	4.00	0.316	6.24	.0126	3.48	0.426	4.63	.0153	3.00	0.527	3.74	.0176	2.52	0.629	3.14	.0198	2.09	0.719	2.74	.0216
"	12:25	10		2040	9.79	8.44	9.11	.00180	2.34	16.5	3.00	.0245	14.22	.00515	.6097	4.01	0.314	6.32	.0124	3.49	0.424	4.68	.0152	3.00	0.527	3.76	.0175	2.54	0.624	3.18	.0196	2.09	0.719	2.76	.0215
"	12:35	10		1200	9.79	8.44	9.11	.00180	2.34	16.5	3.00	.0245	14.22	.00515	.3586	4.01	0.314	6.32	.0124	3.49	0.424	4.68	.0152	3.00	0.527	3.76	.0175	2.54	0.624	3.18	.0196	2.09	0.719	2.76	.0215
"	12:45	10		900	9.77	8.43	9.10	.00179	2.16	16.4	2.97	.0274	14.22	.00571	.2690	3.57	0.408	4.89	.0160	3.11	0.504	3.96	.0185	2.68	0.594	3.36	.0206	2.25	0.683	2.92	.0226	1.86	0.765	2.61	.0244
"	12:55	10		1560	9.75	8.41	9.08	.00179	2.13	16.0	2.91	.0279	14.22	.00570	.4662	3.50	0.409	4.88	.0163	3.05	0.501	3.99	.0187	2.62	0.589	3.39	.0207	2.21	0.673	2.96	.0227	1.82	0.753	2.65	.0245
"	13:05	10		750	9.71	8.38	9.04	.00177	2.08	15.4	2.80	.0287	14.22	.00565	.2242	3.40	0.414	4.87	.0167	2.96	0.500	4.03	.0190	2.55	0.581	3.47	.0210	2.14	0.662	3.05	.0229	1.77	0.738	2.75	.0245
"	13:15	10		450	9.69	8.35	9.02	.00179	2.05	15.0	2.74	.0294	14.22	.00566	.1345	3.30	0.418	4.77	.0172	2.87	0.502	3.97	.0194	2.47	0.579	3.44	.0213	2.08	0.654	3.05	.0231	1.72	0.723	2.76	.0247
"	13:25	10		780	9.65	8.31	8.98	.00179	1.99	14.4	2.63	.0308	14.22	.00569	.2331	3.16	0.428	4.66	.0180	2.76	0.502	3.97	.0200	2.37	0.574	3.47	.0219	2.00	0.643	3.10	.0236	1.64	0.709	2.81	.0251
"	13:35	10		480	9.57	8.23	8.90	.00179	1.86	13.1	2.40	.0339	14.22	.00572	.1435	2.86	0.439	4.54	.0195	2.50	0.499	4.00	.0212	2.15	0.558	3.57	.0229	1.80	0.617	3.23	.0245	1.49	0.670	2.98	.0259
"	13:45	10		330	9.39	8.09	8.74	.00173	1.62	10.5	1.95	.0401	14.22	.00554	.0986	2.38	0.412	5.01	.0211	2.08	0.453	4.56	.0225	1.78	0.494	4.18	.0239	1.50	0.533	3.87	.0251	1.24	0.566	3.63	.0261
"	14:10	40		75	9.06	7.83	8.44	.00164	1.25	6.0	1.12	.0565	13.85	.00457	.0230	1.68	0.297	7.33	.0215	1.46	0.315	6.91	.0224	1.26	0.331	6.58	.0231	1.06	0.347	6.27	.0238	0.88	0.362	6.01	.0245
11-1-41	10:26	30	15	310	9.07	7.92	8.49	.00153	1.77	6.7	1.26	.0316	14.05	.00284	.0938	2.98	0.210	11.1	.0116	2.60	0.243	9.60	.0128	2.22	0.275	8.39	.0140	1.88	0.308	7.58	.0150	1.55	0.338	6.90	.0159
"	10:56	30		310	9.07	7.91	8.49	.00155	1.77	6.7	1.26	.0319	14.05	.00286	.0938	2.96	0.211	10.9	.0117	2.59	0.245	9.40	.0130	2.21	0.279	8.26	.0141	1.87	0.309	7.45	.0151	1.54	0.339	6.79	.0161
"	11:26	30		340	9.06	7.90	8.48	.00155	1.76	6.6	1.24	.0322	14.01	.00285	.1031	2.92	0.213	10.8	.0118	2.55	0.245	9.40	.0130	2.18	0.275	8.28	.0142	1.84	0.308	7.48	.0152	1.52	0.337	6.83	.0161
"	11:56	30		270	9.05	7.89	8.47	.00155	1.75	6.4	1.20	.0325	13.98	.00280	.0821	2.90	0.209	11.0	.0118	2.54	0.240	9.60	.0129	2.16	0.272	8.47	.0141	1.83	0.301	7.65	.0150	1.51	0.326	7.02	.0159
"	12:26	30		240	9.05	7.88	8.46	.00156	1.74	6.3	1.18	.0330	13.94	.00279	.0732	2.87	0.209	11.0	.0119	2.51	0.239	9.57	.0130	2.15	0.270	8.48	.0141	1.81	0.299	7.65	.0151	1.50	0.325	7.04	.0160
"	12:56	30		250	9.05	7.87	8.46	.00157	1.73	6.3	1.18	.0335	13.94	.00283	.0762	2.83	0.212	10.7	.0121	2.46	0.244	9.32	.0133	2.11	0.273	8.33	.0144	1.78	0.301	7.55	.0153	1.47	0.328	6.93	.0162

*Stage at range 15 observed 10 minutes before that at range 0 to compensate for wave velocity.

A great number of these mechanical analyses have been plotted in figure 9. They are classified as to high and low rates (higher or lower than 1,000 lbs/hr.) and compared with a group of bed samples taken in March 1941 throughout the whole measuring reach. Despite the scatter, which probably represents sampling errors, there appears to be little, if any significant differences between the different groups of curves, thus indicating that the mixture for Mountain Creek moves like uniform grains under the prevailing flow conditions. If the bed samples are the same as the transportation samples, particles of all sizes must have the same average velocity, and behave similarly at both high and low stages, which is just the predominant characteristic of uniform grain. The only variable describing transportation in this case, therefore, is the rate of transportation and it seems very probable that the whole mixture can be described by one representative grain diameter such that the rates of transportation are the same as for a bed composed of uniform grains of this representative size.

The hydraulic measurements

As previously mentioned, all velocity measurements are shown in figure 8 with a smooth curve drawn through the plotted points. This curve was used for the determination of the average velocity as a function of the stage in the center of the measuring reach.

The stages at the two ends of the measuring reach are recorded in columns 6 and 7 of table 3. The figures represent the average of the values observed at the beginning and end of the measuring periods. During floods, when the stage changes are relatively rapid, changes in discharge move downstream with the velocity of a small wave. The wave velocity in this case is not high and the changes take a relatively long time to travel the entire length of the measuring reach. This time amounts to something in the order of 10 minutes for low stages. For this reason, during a fast rising flood the discharge may be much higher at the upper end of the reach than at the hopper. In such instances the energy grade line is not the same as the slope of the water surface and a correction must be introduced for "wave action". Unfortunately, the stage recorders do not permit the time to be read with sufficient accuracy for such a detailed correction. The following procedure was adopted to overcome this difficulty.

In addition to the slow changes in discharge, every flood has some wave-like, rather sudden changes in discharge which can be traced downstream from one recorder to the other. As a particular wave passes the various ranges, it is accompanied by the same discharge, and therefore, the corresponding stage, which are usable for the determination of the energy grade line for that discharge. The only partly correct assumption made was that the time interval between corresponding stages is a constant for any one flood. This time has been found to be a little less than 10 minutes. All stage readings taken during floods, therefore,

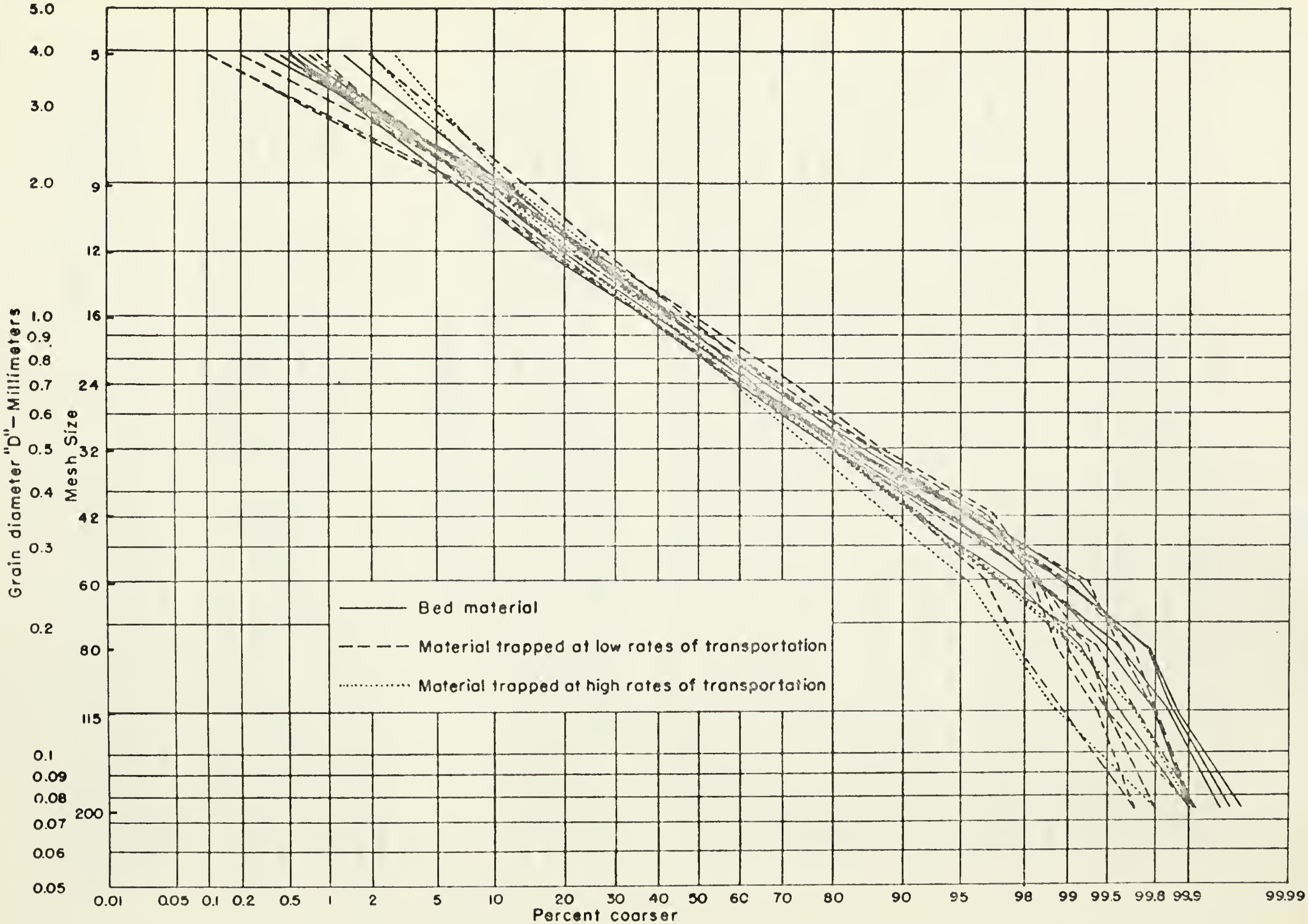

Figure 9–Mechanical composition of sediment in Mountain Creek.

are taken about 10 minutes earlier at range 15 than at range 0. Even for low stages of long duration when a difference of 10 minutes is insignificant, all readings were made at this same 10-minute interval to make the procedure uniform.

Column 8 of table 3 is the average of the readings in column 6 and column 7, and is the stage at the center of the measuring reach. It is this gage reading that is used in figure 8 to determine the prevailing average velocity (column 10), the average area (column 11), and the average wetted perimeters (columns 12 and 14). The average slope (column 9) is calculated by dividing difference of the two gage readings (columns 6 and 7) by the distance of 750 feet. Column 4 gives the temperature of the water in degrees C.

APPLICATION OF FORMULAS

From the data given in table 3, the rate of transportation, discharge, and slope could be plotted as a function of stage (column 8) as was done in figure 8 with the mean velocity. The points would more or less closely follow a curve, but would have no general significance. Since no general conclusions could be obtained from such plots the graphs are not included in this report.

The problem of Mountain Creek was defined in the introduction as an extension of flume studies to natural rivers. It is, therefore, logical to use all possible information from past flume studies to express the results of river measurements in the most general way, making them comparable with all available information. A bed-load formula, in general, expresses the rate of transportation as a function of the discharge and of the physical characteristics of moving sediment. For simplicity in the analysis of the problem both the transportation and discharge are expressed as a rate per unit width of the bed. This reduction to a unit basis is fairly simple for the transportation, but probably requires some explanation for the discharge. It is general usage in friction formulas to describe the size of a cross section by means of the hydraulic radius. It has been shown (2, 3) that the hydraulic radius, $R = A/P$, numerically equals that part of the cross-sectional area pertaining to a certain unit length of the wetted perimeter. Each unit length of the wetted perimeter with its corresponding area forms a unit for the dissipation of energy insofar as the energy of the water in the area is transformed into turbulence along the wetted perimeter. The roughness of the boundry determines its ability to dissipate energy; therefore, when the roughness varies over a cross section, it is evident that the rougher parts will dissipate more energy. Inasmuch as each square foot of cross-sectional area contributes a like amount of energy to the dissipation along the boundary, a rougher part of the boundary will utilize per unit length the energy from a larger

part of the area than a smooth part. Therefore rougher parts of the boundary have relatively greater hydraulic radii. It is often assumed that friction along the banks can be neglected if the width of the section is greater than 10 times the depth. The contrary has been found for Mountain Creek where the banks are relatively much rougher than the bed.

It has also been proved in the analysis of flume experiments (3) that the transportation of bed load depends on the dissipation of energy and that only that part of the energy is effective that is transformed into turbulence along the bed. It is evident, therefore, that the question of "side-wall effect" is as important for bed-load transportation as for the friction itself.

Side-wall effect

Any friction formula such as Manning's gives the energy loss of a stream along certain solid boundaries. As Manning's formula is one of the best and most convenient for rough boundaries, it will be used in this report. In Mountain Creek the water is in contact with the banks as well as with the bed and consequently energy is dissipated along both. It has been found that the friction coefficients governing the energy dissipation along the different parts of a wetted surface are independent of each other. Based on this fact, the writer has developed a method for distributing the energy loss among the parts of the wetted perimeter with different roughness (2) (3) and this method will be used to determine the friction along the stream banks that does not contribute to the movement of bed load. The application of the method is the same as in flume experiments except that in a natural stream the banks usually are not vertical.

In describing the method the following symbols are used:

n_b = Manning's n for the bed

n_w = Manning's n for the banks

P_b = The wetted perimeter of the bed

P_w = The wetted perimeter of the bank

R_b = The hydraulic radius of the bed

R_w = The hydraulic radius of the bank

A_t = The area of the cross section

A_b = Part of the area A_t pertaining to the bed

A_w = Part of the area A_t pertaining to the bank

V = Average velocity of the water in the cross section

S_e = Slope of the energy grade line

The following equations express the relationships existing among the variables during any measuring period:

$$A_t = A_b + A_w$$

$$R_b = \frac{A_b}{P_b}$$

$$R_w = \frac{A_w}{P_w}$$

$$V = \frac{1.486}{n_b} R_b^{2/3} S_e^{1/2}$$

$$V = \frac{1.486}{n_w} R_w^{2/3} S_e^{1/2}$$

Of the 11 variables in these equations the following five were determined for Mountain Creek and are given in table 3:

A_t in column 11
P_b in column 14
P_w in column 12
V in column 10
S_e in column 9

If one more variable is known the remaining five can be calculated. The procedure used to overcome this difficulty was to compute sets of values for the other variables using different assumed values for the roughness of the banks, n_w, and compare the results with known values derived from flume experiments. The calculation for the different values of n_w are found in the following columns of table 3:

for n_w =	0.068	columns 17-20
	0.062	21-24
	0.056	25-28
	0.050	29-32
	0.044	33-36

The friction formula

Roughness, in general, cannot be described by a single figure. In addition to the size of the roughness-unit, its form and distribution over the surface are also important factors affecting the ability of a cross section to transform energy. In the case of a granular bed, however, all roughness units have similar form and they cover the entire surface. In this case, the assumption seems reasonable that the size of the particles alone determines the roughness coefficient.

Strickler was probably the first to discover that Manning's n is proportional to the 1/6th power of the diameter of the grains for a granular bed without bed-load movement (5). By measurements in natural streams, he found that the value $n/\sqrt[6]{D}$ (where D is the grain diameter), would increase when transportation begins. He assumed that a change in the position of the single particles in the bed caused this change. Another explanation would be that the transportation itself consumes some energy and thus gives an apparent increase in the roughness. In this case the value $n/\sqrt[6]{D}$ should be a rather continuous function of the intensity of transportation.

This part of the transportation problem is even less solved than the transportation equation itself, mainly because it requires a very precise measurement of the slope. Inability to meet this required precision is a weakness common to most flume studies. In the Mountain Creek study it will be given equal attention with the transportation equation, especially because the great length of the measuring reach permits a reasonably dependable slope measurement.

If the values A_b, A_w, R_b, and R_w, are eliminated from equations 1-5, the following equation is obtained:

$$\frac{A_t \cdot S_e^{3/4} \cdot 1.486^{3/2}}{V^{3/2}} = P_b\, n_b^{3/2} + P_w\, n_w^{3/2}$$

During each measuring period all values in this equation with the exception of n_b and n_w can be measured. This equation gives a straight line if $n_b^{3/2}$ is plotted against $n_w^{3/2}$. The curve for each measuring period can be easily established if $n_w^{3/2}$ is computed for $n_b = 0$, then $n_b^{3/2}$ computed for $n_w = 0$. These values are shown in columns 13 and 15 in table 3 and are plotted in figure 10. It will be noted by a careful examination of these curves that they become relatively flat as the stage and rate of transportation increase and tend to be relatively steep for the low stages.

Except for the few curves indicated by dashed lines, which apply to stages after the crests of the two artificial floods, it is observed that all curves pass through or near the point $n_w^{3/2} = 0.0176$ and $n_b^{3/2} = 0.00132$ or $n_w = 0.0680$ and $n_b = 0.0120$. Does this point have any significance? Before an attempt is made to answer this question, the significance of any one of the curves in figure 10 must be determined. As previously mentioned $n_w^{3/2}$ is a measure for the bank roughness and $n_b^{3/2}$ is a measure of the bed roughness. Each one of the

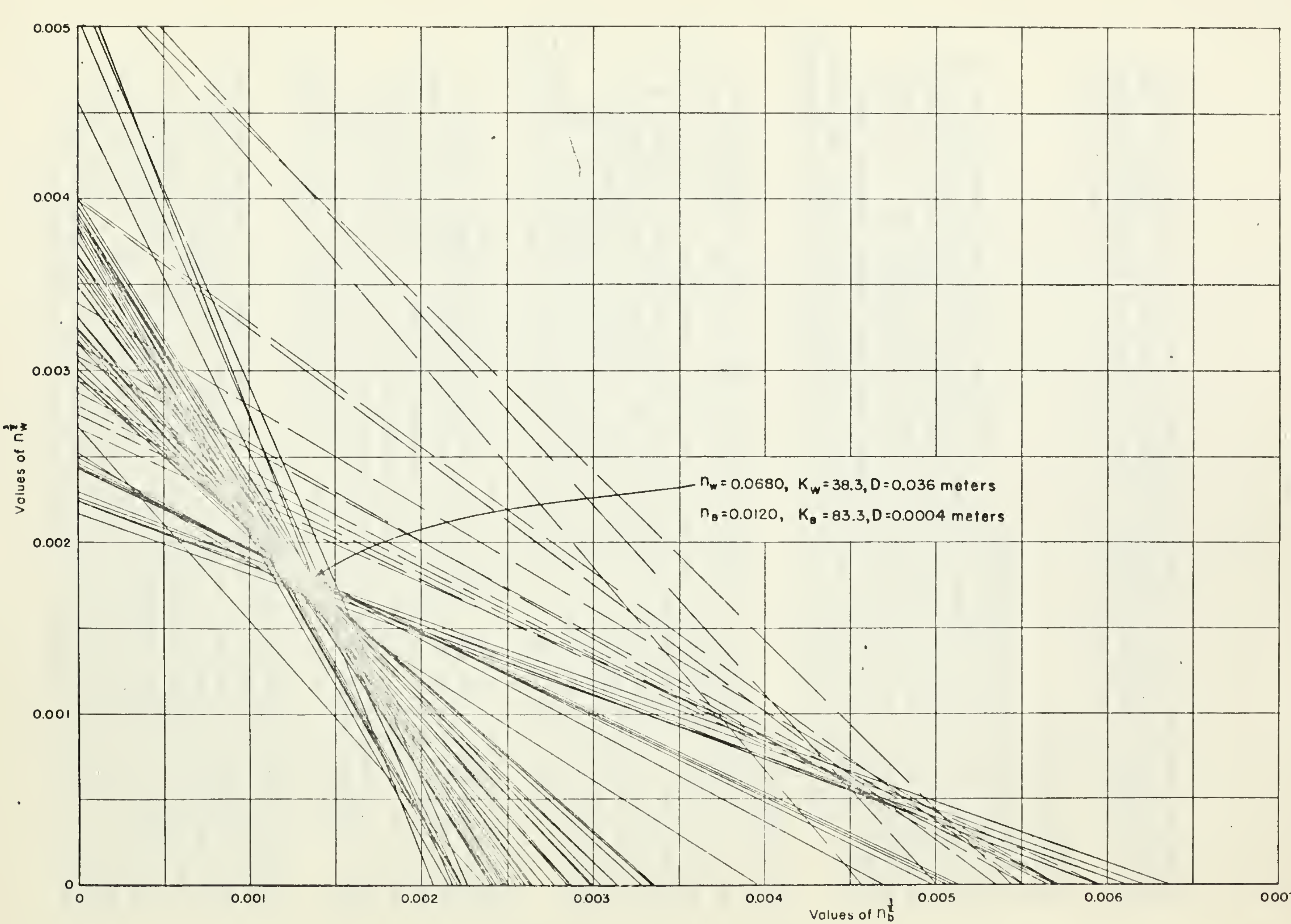

Figure 10–Relation between $n_w^{3/2}$ and $n_B^{3/2}$

curves, therefore, indicates the combinations of values that are possible according to this one set of measurements. It must be noted, however, that only one of all these combinations of values is the true value; hence, if all lines pass through one point, and if the roughness coefficients are both constants, this point definitely represents the true relation. In the Mountain Creek study it is not known whether the roughness coefficient of the bed is a constant or a variable. It is safe, however, to assume that n_w is a constant, and it is known rather definitely from other experiences that the roughness of the bed will not decrease with increasing transportation.

A constant value of n_w is found on any horizontal line in figure 10. The true values, therefore, must concentrate on one horizontal line. If n_b is constant as previously mentioned, $n_w = 0.068$ and $n_b = 0.012$, but if n_b increases with increasing transportation, it is readily seen that n_w must be smaller than 0.068, because only below this point will high rates give greater values of n_b than low rates. This is the reason why in columns 17-36 in table 3, the various data have been completed with assumed values of n_w which range from 0.044 to 0.068.

It has been found in former flume studies with material of uniform grains (5) that the value of $n/\sqrt[6]{D}$ for zero transportation is 1/24 when D is measured in meters or 1/51.7 = 0.0194 when D is measured in cm. It has been found that this same rule can be applied to natural sand mixtures if the diameter, D, is selected so that 65% of the material is finer than D. For Mountain Creek this diameter according to figure 9 is 0.110 cm. The friction factor of the bed, therefore, is expected to be $n_b = 0.0194\sqrt[6]{0.110} = 0.0134$. As this value is in the neighborhood of $n_b = 0.0120$ found for $n_w = 0.068$, the true value of n_b is probably in that vicinity.

The same experiments that led to the constant for the determination of the roughness according to Strickler (and which is slightly different from Strickler's own value) also showed that this factor changes continuously with increasing rates of bed-load transportation. A strictly empirical curve has been derived from these experiments and gives the change of the factor in terms of transportation. This curve is compared with the resulting roughness factors for different assumed values of n_w in figure 11, and is discussed below, together with the presentation of the transportation measurements.

Bed-load formula

A number of bed-load formulas, varying in range of applicability and accuracy, have been proposed. Some of them are strictly empirical, while others are based on some theoretical background. If all of these formulas are used in an attempt to estimate the rate of transportation

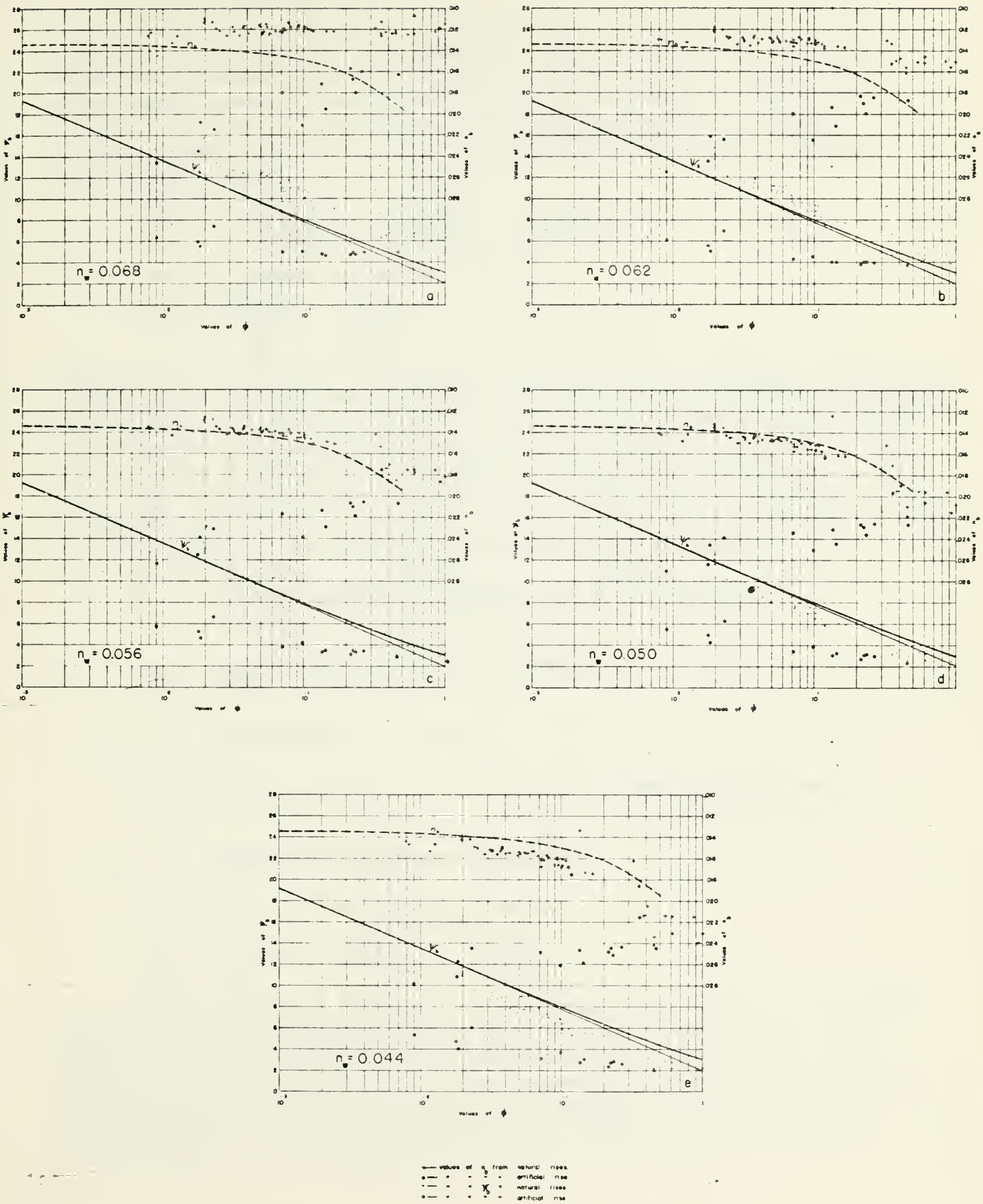

Figure 11- Variation of n_b and ψ_b with rate of transportation for various assumed side-wall roughnesses

for a given flow condition the results vary within unreasonably wide limits. The writer, after reviewing a large number of representative experiments, has proposed the $\Psi-\Phi$ bed-load formula (3). This relationship seems to concentrate a maximum of available data into a single curve, and is used for comparative purposes in analyzing Mountain Creek. In the $\Psi-\Phi$ relationship the intensity of bed-load transportation

$$\Phi = \frac{1}{F} \frac{q_s}{(\rho_s-\rho_f)g} \sqrt{\frac{\rho_f}{\rho_s-\rho_f}} \frac{1}{g^{1/2} D^{3/2}}$$

is plotted against the hydraulic variable

$$\Psi = \frac{\rho_s-\rho_f}{\rho_f} \frac{D}{S_e R_b}$$

Φ can be interpreted as the ratio of the velocity $\frac{q_s}{(\rho_s-\rho_f)gD}$ the bed load would have if it moved down the river as a solid layer of the thickness D (grain diameter), divided by the settling velocity $F\sqrt{gD\frac{\rho_s-\rho_f}{\rho_f}}$ of the sediment particle in water. Ψ can be interpreted as the ratio of the weight of the grain $(\rho_s-\rho_f)gD^3$ divided by the friction of the flow $\rho_s S_e R_b D^2 g$ acting on the portion of the bed covered by the particle. (Disregarding certain numerical constants of the order of magnitude equal to unit, which must be determined as statistical averages).

F = Factor in Rubey's equation for the settling velocity (0.816 for Mountain Creek sand) (3)

q_s = Rate of transportation in weight under water per foot of width

g = Acceleration of gravity

ρ_s and ρ_f = Density of the solid sediment and the fluid, respectively

D = Representative grain diameter

S_e = Slope of the energy gradient

R_b = Hydraulic radius of the bed

From this short interpretation of the two parimeters Ψ and Φ one might get the impression that the bed-load formula is an equation between averages like most hydraulic equations. In reality, this formula is derived as a relationship between the statistical movement of sediment grains and the statistical velocity distribution in a turbulent flow. This fact explains the rather complicated shape of the curve.

The intensity of transportation Φ, shown in column 16 of table 3, is not dependent on the choice of the side-wall friction. The value of Ψ has been calculated for the 5 different assumed values of n_w and appears in the columns 19, 23, 27, 31, and 35. For this latter calculation D = 0.68 mm. was introduced, which corresponds to the sieve size through which 35% of the mixture passes (3). The hydraulic radius of the bed, R_b, is introduced because only flow along the bed is responsible for transportation. In the lower part of the diagrams in figure 11 the values of Ψ are plotted against the intensity of transportation Φ. The fact that the same intensity of transportation enters both the relationships for transportation and friction emphasizes its importance and shows the close relation between the two laws.

Each one of the 5 graphs shown in figure 11 represents the n_b and Ψ values for all measuring periods calculated for one of the assumed values of n_w. For comparison the curves are shown which summarize the results derived from flume experiments. A review of these diagrams shows that for n_w = 0.068 all significant points are above the flume experiment curves. For n_w = 0.062 the points are closer to the curves but still too high. For n_w = 0.056 the n_b values are slightly above the line, but Ψ appears to fit the curve reasonably well. For n_w = 0.050, and especially for n_w = 0.044, the points seem distinctly too low, but n_w = 0.056 appears to represent the true value or at least is closest to the curve, consequently, this value of n_w is used in all subsequent computations of the Mountain Creek data.

In each of the diagrams in figure 11 some of the points are marked by a circle. They represent measurements during the rapidly falling stages of the two artificial floods. During these stages, a distinctly abnormal flow took place, that is, the average velocity, the slope, and the rates of transportation are completely out of harmony with the results of the flume experiments. This lack of harmony did not occur during or after any natural flood and must be caused by the sudden decrease in discharge that occurred during the artificial floods. For this reason, these points are discarded as not characteristic for natural flows in a natural river.

It must be noted, however, that a similar deviation of the points has been observed in some of the flume studies conducted at the U. S. Waterways Experiment Station, Vicksburg, Miss. These experiments have been discussed previously (3) and it was shown that Ψ for these experiments is low, as is the case for the artificial floods on Mountain Creek. A simple calculation reveals that the beds in both the flume and in Mountain Creek were too rough as compared with the normal condition. For the explanation of these anomalies, it will be well to bear in mind how they developed in Mountain Creek.

The configuration of the surface of the bed, the distribution of the different grain sizes on the bed, and a certain stratification in the top layers seem to change characteristically with the stage. If the stage drops suddenly from a comparatively high value to a low value, accompanied by very small rates of transportation, the intermediate stages will not have time to change the structure of the bed and they will all flow over a bed whose configuration is characteristic of the bed during a higher stage. The relative characteristics of a high-water bed and a low-water bed cannot be definitely described because of the impossibility of collecting bed samples during flood stages.

Concluding the discussion of the application of known formulas to the measurements in Mountain Creek, the following statements can be made:

(1) The rate of transportation in Mountain Creek can be described by the $\psi - \Phi$ method and follows the same curve as flume experiments with uniform grain size if the roughness of the banks is assumed to be $n_w = 0.056$ and if a grain diameter, which corresponds to the sieve through which 35 percent of the mixture passes, is introduced as representative.

(2) The roughness of the bed can be described by Strickler's formula if that sieve size through which 65 percent of the mixture passes is introduced as the representative diameter for roughness. The roughness of the bed increases with increasing intensity of transportation according to the same curve that has been derived from flume experiments with uniform grains.

APPLICATION OF THE RESULTS

In the preceding discussion the characteristics of the flow and bed-load transportation in Mountain Creek were described as accurately as possible, and it was found that three equations or principles that govern the flow and transportation on a uniform movable bed as derived from flume experiments were applicable to this stream. They are:

(1) The equation for transportation of bed load

(2) The equation for the roughness of a movable bed as represented by the $n_b - \Phi$ curve.

(3) The equation for the distribution of the energy spent by the flowing water between the different parts of the wetted perimeter, if the different parts have different roughness.

The accord between the equations derived from flume experiments and the measurements in the creek is so close over the whole length of the curves that it cannot be a mere coincidence. It seems rather safe, therefore, to assume the applicability of the curves not only for the measuring stretch in Mountain Creek, but also for other flows of similar character. Thus, it is possible to calculate the capacity for transportation of any similar stream using merely a description of its bed by cross sections and a mechanical analysis of the bed material.

The Mountain Creek slopes for each measuring period were measured individually and the average velocity in terms of stage was determined by direct measurement. In the case of other streams where these measurements are not available, another method must be devised. This can be accomplished by the use of an "idealized section" with a level bed, the average bed slope for the energy gradient at all stages and velocities calculated by the friction formula. A roughness factor for the banks, of course, must be assumed.

To summarize the Mountain Creek data twin-diagrams are used which show rate of bed load transportation and water depth as functions of the discharge. In such diagrams the stage-discharge relation indicates the tendency toward channel flooding and the discharge-transportation relation indicates the tendency toward change in the bed. These two conditions underlie the most important problems to be studied on any reach of a river.

To illustrate the general principles involved in the calculations of the hydraulic and sedimentary characteristics of a reach of a river, the Mountain Creek data will be discussed and then the method will be used to study the effect of changes in the different characteristics for the same reach. By this method it is possible to study separately the influence of different characteristics such as width of the bed and roughness of the banks, and also to compare the effectiveness of different artificial controls in changing the flow conditions.

The idealized section

The cross section as defined by the curves in figure 8 is rather complicated and cannot be determined without extensive measurements. It is, therefore, replaced by an idealized section with a horizontal bed of the same width as the average section (14.22 ft.), banks sloping 1:1, a roughness of $n_w = 0.056$ previously calculated (figure 11) and an overall slope of $S = 0.00150$.

The main dimensions of the section are:

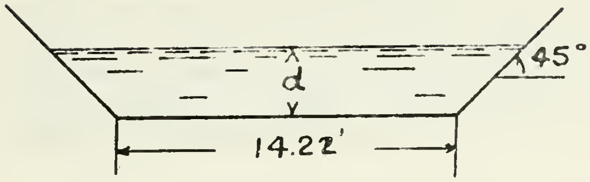

Figure 12.—The idealized section for Mountain Creek.

$n_w = 0.056$

d = water depth in ft.

Area = $d\,(14.22 + d)$ sq. ft.

$P_b = 14.22$ ft.

$P_w = 2.83\,d$ ft.

$S = 0.00150$

Grain diameter for transportation = D_{35} = 0.068 cm.

Grain diameter for roughness = D_{65} = 0.110 cm.

The following formulas are used:

$$\Phi = \frac{1}{F}\left[\frac{Q_s}{D^{3/2}}\right]\frac{1}{B g(\rho_s-\rho_f)}\sqrt{\frac{\rho_f}{\rho_s-\rho_f}}\,\frac{1}{\sqrt{g}} = \frac{Q_s}{3320}$$

Here Q_s is the total transportation in pounds under water per hour

$$R_b = \frac{\rho_s-\rho_f}{\rho_f}\,\frac{D}{S\,\Psi} = \frac{2.38}{\Psi}\ ft$$

Then

$$V = \frac{1.486}{n_b} S^{1/2} R_b^{2/3} \text{ is used}$$

where n_b is determined from figure 11 for different values of Φ, and

$$R_w = \left\{\frac{V.n_w}{1.486.S^{1/2}}\right\}^{3/2}$$

The wetted area, $d(14.22 + d) = 14.22\,R_b + 2.83\,d\,R_w$

or $d^2 + (14.22 - 2.83\,R_w)\,d - 14.22\,R_b = 0$

gives the water depth d; that is,

$$d = -\frac{(14.22-2.83\,R_w)}{2} \pm \sqrt{\left(\frac{14.22-2.83\,R_w}{2}\right)^2 + 14.22\,R_b}$$

and

$$Q = V.d\,(14.22 + d)$$

The calculation of the discharge-stage-transportation relationship for any such section by the above equations can only be made by the trial-and-error method unless the calculation is started by assuming values of the transportation. For various rates of transportation the values of Φ can be calculated and Ψ and n_b determined from figure 11. The other terms, R_b, V, R_w, d and Q can then be calculated easily in this sequence. Table 2 gives this calculation for six values of assumed rates of transportation.

Table 2.—Relationship of Q_s, d and Q applying to the idealized section

Q_s	Φ	Ψ	R_b	n_b	V	R_w	d	Q
Lbs./hr.*			Feet		Ft./Sec.	Feet	Feet	C.f.s.
10	0.0030	16.5	0.144	0.0135	1.18	1.23	0.18	3.06
30	0.00904	13.75	0.173	0.0137	1.30	1.42	0.24	4.51
100	0.0301	10.8	0.220	0.0140	1.50	1.76	0.33	7.20
300	0.0904	8.2	0.290	0.0148	1.70	2.13	0.47	11.7
1000	0.301	5.40	0.440	0.0175	1.90	2.51	0.79	22.5

*Under water.

A plot of Q_s and d against Q, the two most important curves, is shown in figure 13. For comparative purposes, the experimental data from table 3 are also shown in figure 13. As might be expected, the measurements show a small systematic deviation from the curves in figure 13 but still this deviation is permissible for practical purposes and is definitely smaller than the errors involved in estimating the duration of various discharges for some period of time in the future. Again the measurements made during the falling stages of the two artificial floods give the greatest deviations and may be discarded for practical use as previously explained.

As calculations based on this idealized section give results that compare satisfactorily with actual measurements, it will be used to study the influence of changes in the cross section on the capacity for both water and bed load. As a first example, the influence of the width of the bed is discussed because it is generally assumed to have a strong influence on the carrying capacity of a stream.

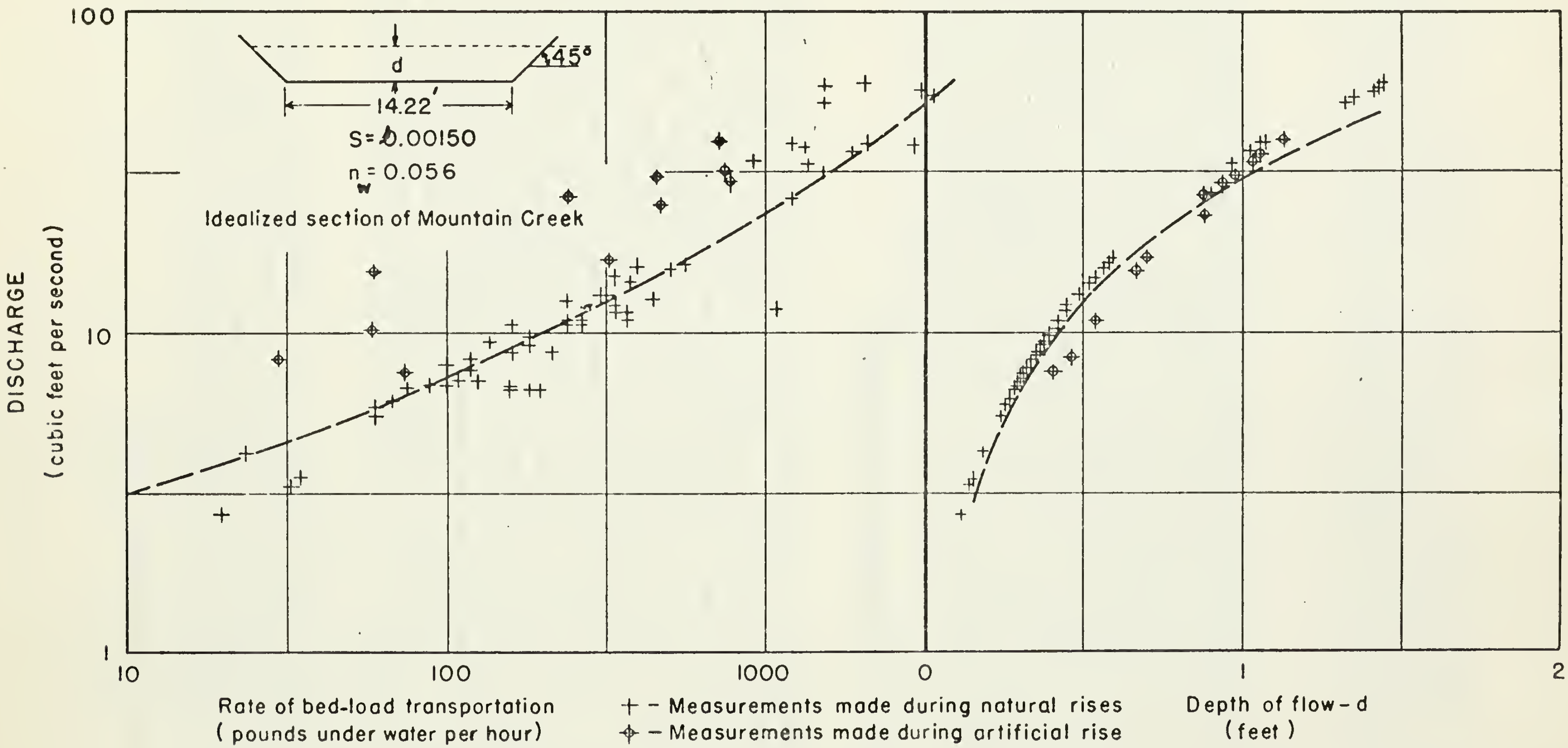

Figure 13—Measured values of bed-load transportation and depth of flow compared with values calculated for an idealized section. Mountain Creek, Greenville, S.C.

Influence of the width of the bed

The concentration of water into a narrower channel is generally supposed to increase its ability to move bed load. In general this statement is true but in our special case it is not. Figure 14 shows rate of transportation and stage plotted against discharge for bottom widths of 10.22, 14.22, and 18.22 feet, determined by a computation procedure similar to table 2.

As expected, it is seen that the stage for any one discharge is higher for a narrow bed. The narrow bed definitely increases the danger of flooding. The carrying capacity, however, does not seem to be influenced very greatly and the curves intersect, thus indicating a higher carrying capacity for narrow beds at low stages and for the wide bed at high stages. Why does this section behave normally at very low stages, but quite abnormally for higher stages? The explanation is found in the roughness of the banks which is rather high compared with the width of the bed. The higher the stage the more area of bank surface is submerged. Being relatively rougher than the bed, the banks absorb an increasing part of the total energy of the water as the stage increases. The discharge-depth curves indicate a definite increase of depth for any one discharge with decreasing width, thus increasing the percentage of rough bank surface which counteracts the effect of the narrow channel so far as transportation is concerned. Further investigation, therefore, will be concentrated on the bank friction where the greatest losses appear to occur.

Influence of the roughness of the banks

In figure 15 relations between discharge and the rate of transportation and stage are shown for various sections which differ only in the roughness of the banks. The values for n_w of 0.068, 0.056, and 0.044 represent banks of different types of vegetative cover while $n_w = 0.0134$ represents a bank of the same roughness as the bed, a value that a rather smooth concrete probably would have. By decreasing n_w from 0.068 to 0.044, the transporting capacity is increased about 2.5 times, decreasing the stage some at the same time. An improvement of the bank cover represents, therefore, an ideal remedy for deposition in the bed, while the introduction of artificial roughness by planting of trees or shrubbery will decrease the danger of scour in the bed. This rule is rather general. The extent of the influence, however, should be checked in each case as different sections will react to a different degree.

The roughness of the banks, n_w, is a very important factor affecting the capacity of a stream to transport bed load, in fact, much more important than has generally been realized. It is very unfortunate that so little information is available in the literature as to values to use for the roughness factor, n, when applied to banks as a separate part of

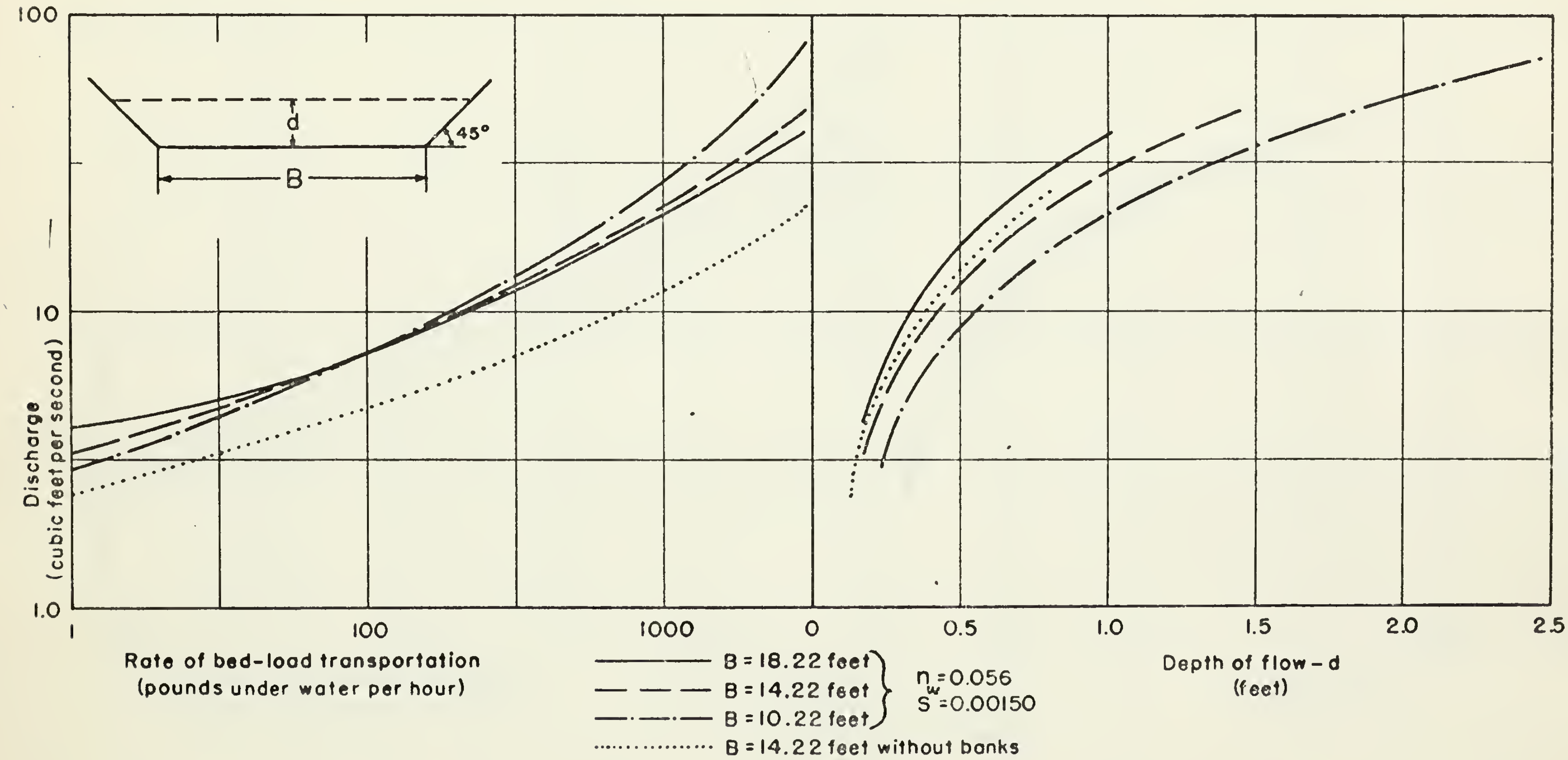

Figure 14-Channel capacity for water and bed load for various bed widths

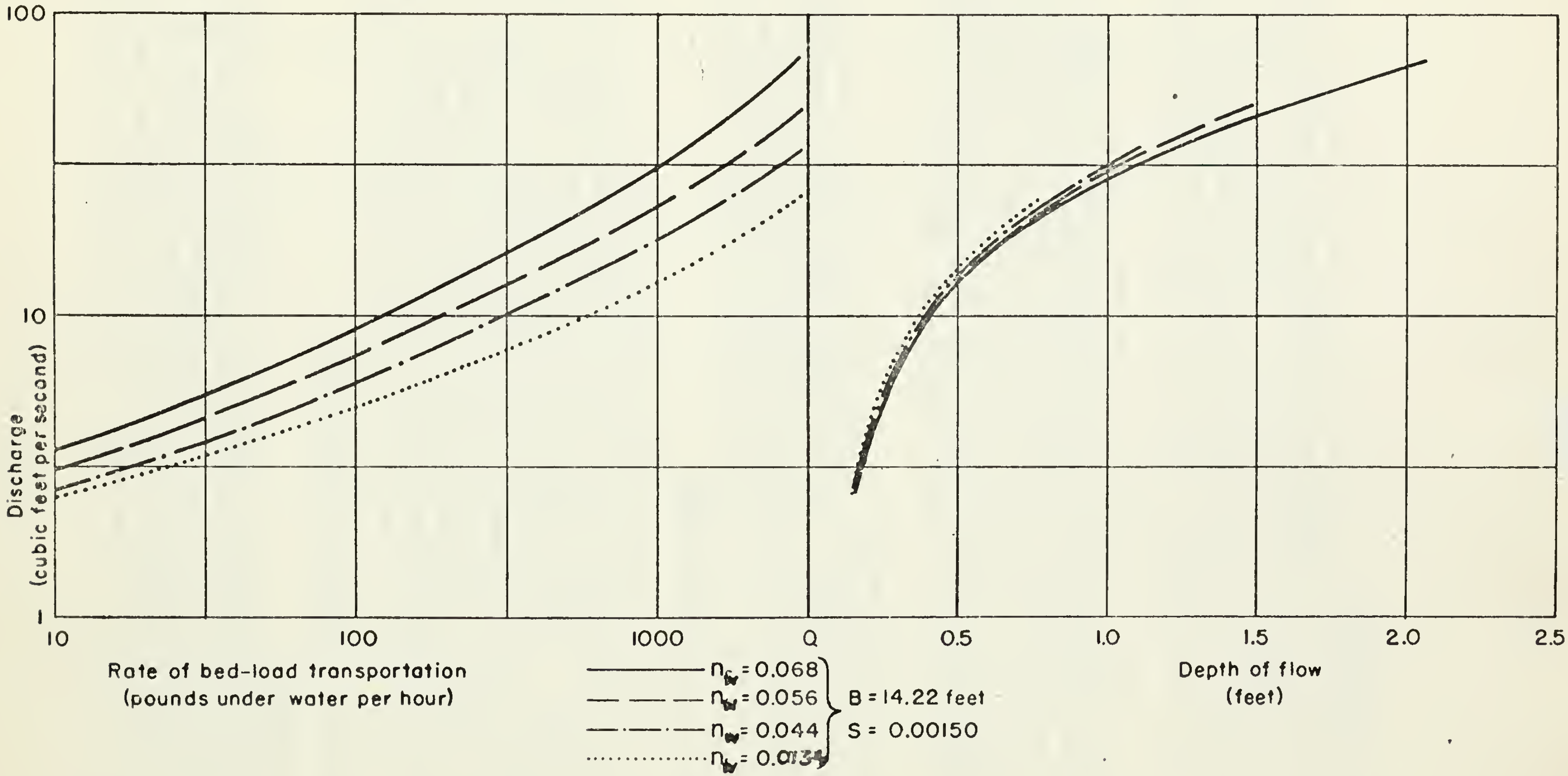

Figure 15—Channel capacity for water and bed load for various roughnesses of the banks.

the wetted perimeter. These values are more difficult to obtain than those for overall roughness for the whole cross section but are of such significance in the computation of bed-load transportation that they must be obtained in some manner. The determination of values of n for different conditions of vegetal cover that can be applied to the part of the wetted perimeter affected is a research study worth of serious attention.

Influence of the steepness of the banks

The question as to whether steep or flat banks are the most effective in influencing the movement of water and bed load is answered in figure 16 where the curves of depth and rate of transportation for vertical banks are compared with those for slopes of 1:1 and 1:2. It is noticeable that the change in bank slope from 1:2 to 1:1 will double the transporting capacity while the increase from 1:1 to vertical is not so pronounced. A short analysis shows that there is a most efficient slope somewhere between vertical and 1:1 which, however, for most cases is too steep to be stable. As a rule for any natural section, the steepest bank will give the highest carrying capacity.

Influence of the general slope

As the expression "general slope" may indicate, this discussion of the application of the results deals only with uniform flows. In uniform flow the slope of the bed, the water surface, and the energy grade line are identical. It is possible under these conditions to determine stable conditions or the stable end-condition toward which changes in the bed develop.

From this point of view, it is interesting to study the influence of a changed slope using the stable slope that the river will assume eventually if it is not disturbed otherwise to measure any tendency to scour or deposit. The effect on the depth and rate of transportation of a 10 percent increase or decrease in the slope is shown in figure 17. This figure shows that the total 20 percent increase will about double the transporting capacity. The 20 percent difference in slope represents a change of 0.0003 or 1.5 ft/mile in the case of Mountain Creek.

It is now possible to measure the effect of a change in slope on the condition of the banks. The procedure is to determine the change in slope that would just counteract the change in transportation capacity due to the change in the bank condition. For example, a change in the steepness of the banks from 1:2 into 1:1 would call for a decrease in the slope of about 20 percent or 1.5 ft/mile, and a change of the bank roughness from 0.044 to 0.068 would call for an increase in the slope of about 2.0 ft/mile.

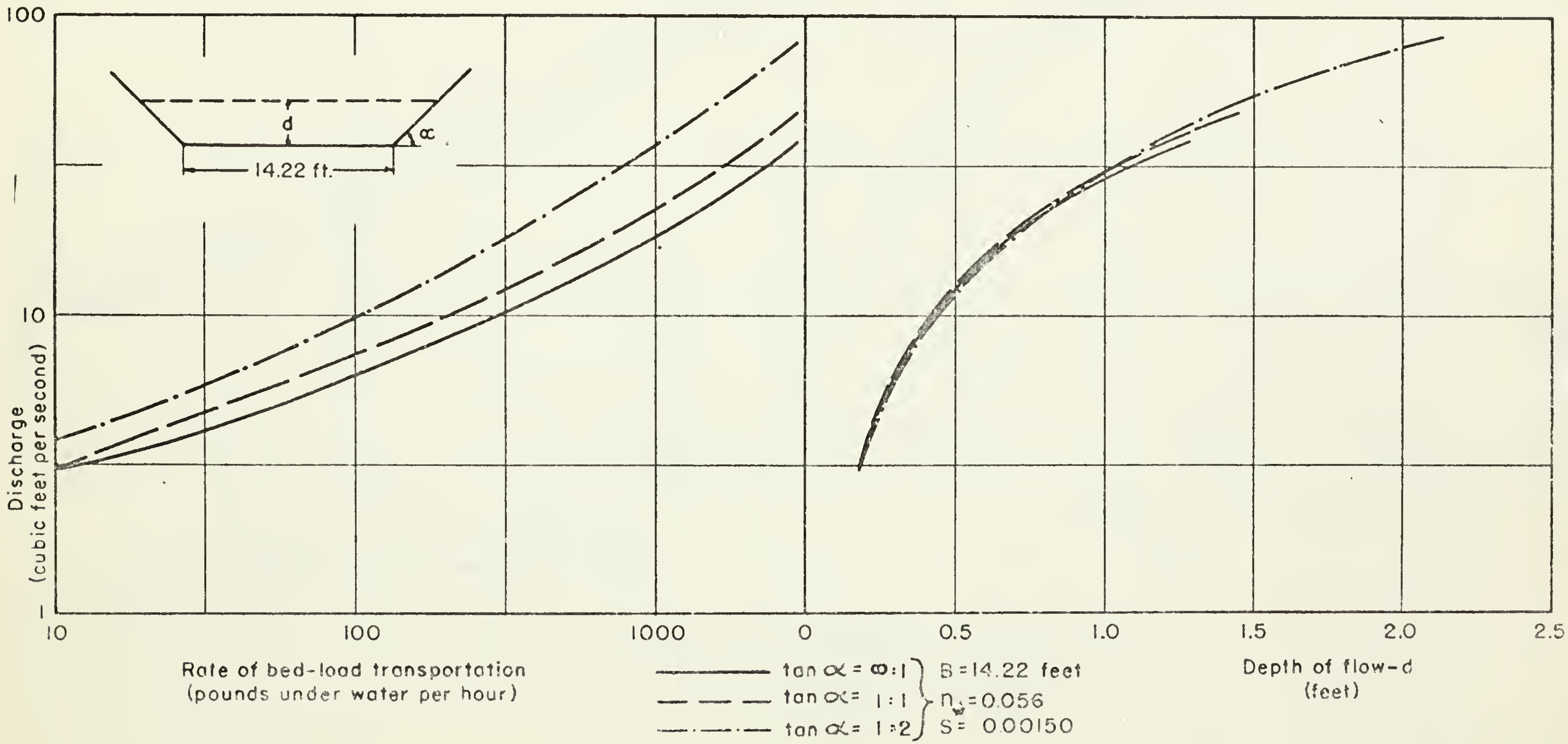

Figure 16- Channel capacity for water and bed load for various slopes of the banks.

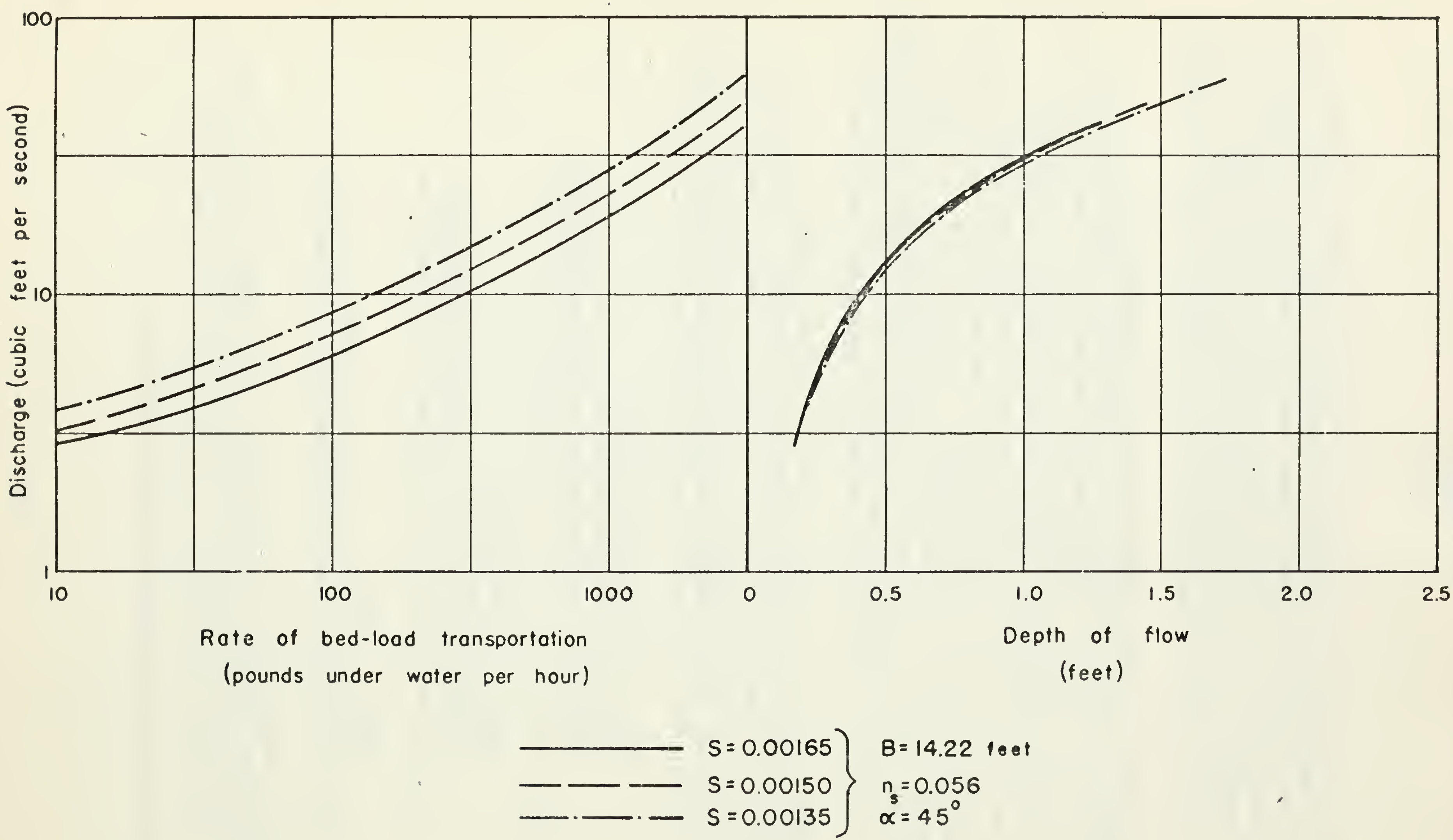

Figure 17 - Channel capacity for water and bed load for various slopes.

Application of the idealized section in general

The analytical method of treating bed-load problems consists of the simultaneous application of the three equations or principles, stated at the beginning of "Application of the Results" and, for convenience, repeated here:

(1) The equation for the transportation of bed load, giving the rate of bed-load transportation in terms of flow.

(2) The equation for the roughness of a movable bed, as represented by the $n_b - \Phi$ curve, giving the friction factor along the bed in terms of the rate of transportation.

(3) The equation for the distribution of the energy spent by the flowing water between the different parts of the wetted perimeter, if the different parts have different roughness.

All three principles must be applied in any future problem. However, it has not yet been proved that the same equations can always be used or that they can always be applied in the same manner. For example, it is conceivable that in a certain stream the assumption of a horizontal bed will entirely change its character. In this case, a different cross section must be used as representative.

The basic, but unproved, assumption underlying the entire procedure is that the transportation of bed material always occurs according to a fixed law or, in other words, that the river is always carrying bed sediment to capacity. In many cases this assumption is true and only in these cases is the method applicable. If the supply of sediment in general is insufficient or even if only some particular grain sizes of the mixture are lacking, the effective grain diameter may change and the problem becomes hopelessly complicated and uncertain, because then transportation depends not only on the amounts of the discharges but also on their sequences. Unfortunately, no method is now available for verifying the applicability of the transportation formula except by checking against direct measurement of the transportation.

If the rate of transportation plotted against stage or discharge (eliminating any short-time fluctuations by averaging) forms a smooth curve, changes in a river can be predicted by the above method. If the measurements deviate systematically from the curve, these deviations must be considered and in most cases they will complicate the analytical method considerably. A systematic deviation of this kind took place in Mountain Creek directly after the two artificial floods which were

created by opening the gates of the pond upstream. This deviation seems to have been caused by the sudden decrease of the discharge which was so rapid that sufficient time was not available for the bed to readjust its configuration to the conditions of the lower flow. Fortunately, this phase is not important in this creek, as it does not occur after natural floods. The findings from experiments now in progress (see Appendix I) are expected to be of considerable assistance in eliminating some of these uncertainties affecting the solution of bed-load problems.

CALCULATION OF THE AVERAGE ANNUAL TRANSPORTATION

In the preceding sections it has been shown that for a sufficiently long and uniform stretch of a river there exists a relationship between the discharge and the rate of bed-load transportation and that this relationship can be determined by direct measurement or by the application of certain formulas. This rate is called the transporting capacity of the stream. A stream will remain stable as long as it is able to adjust the incoming load to its capacity by scour or deposition without excessive change in its bed. Therefore, continuous deviations in one direction from the capacity will finally result in a change of the profile while alternating deviations of much greater extent may leave the sections generally stable. This latter case is found rather frequently when the adjoining stretch upstream has a lower capacity for high stages but a higher one for low stages or vice versa. To compare the average capacities of these two sections it is necessary to introduce in one way or another the frequency with which the different discharges occur. This frequency can be given as a hydrograph from which a curve for the rates of transportation over time is derived, then by integration of the difference of the rates for the two sections the comparison is obtained. This procedure is very time consuming and usually fails because of the lack of a satisfactory hydrograph.

Instead, the duration curve of the discharge for an average year can be used. The duration curve as used in the planning of hydroelectric power plants represents an ideal tool for studying the integrated effect of certain variables that are functions of the discharge but independent of the sequence of the discharges. The rate of transportation is one of them. There are two main reasons for the use of the duration curve:

(1) The duration curve is very regular and, for that reason, can be sufficiently described by from 6 to 10 points. The integration of the duration curve of the transportation can be executed in from 10 to 20 intervals for the same reason, while the hydrograph of a single year can only be described satisfactorily by hundreds of points.

(2) The duration curves for different streams or different sections of the same stream are extremely similar, allowing the prediction of this curve with reasonable accuracy even without any local measurements, as will be shown subsequently.

Construction of flow-duration curves[1/]

On streams where a record of discharge is not available for the construction of a flow-duration curve, estimates must be based on the curves from adjacent streams. In order to determine whether the flow-duration curves are similar for streams in a particular region where climate and topography are rather uniform, a study was made of streams in the North Carolina Piedmont. This region was selected for study because discharge records of 10 years or more were available in U. S. Geological Survey Water-Supply Papers and other publications for a large number of small streams whose drainage areas are in the range of the size of the streams on which the application of the law of bed-load transportation is permissible. The discharge records for all streams were expressed in mean daily flows.

A flow-duration curve is computed in several steps. First, for the entire period of record a tally is made of the number of occurrences of discharge within certain limits. The percent of time that the discharge prevails in each class limit is next computed and the cumulative percentage then obtained. This procedure is illustrated in the accompanying table 4 for Little Sugar Creen near Charlotte, N. C. (13).

Column 1 of the table shows the limits for grouping the values of discharge. Column 2 shows the number of days that the discharge was within the particular group. Thus, of the 5,477 days of record (1925 - 1939, inclusive), on 3 days the average daily discharge was between 0 and 1.9 cubic feet per second; on 94 days the discharge was between 2.0 and 3.9 c.f.s.; on 483 days the discharge was between 4.0 and 6.9 etc. Column 3 shows the percent of time that the discharge prevailed in the various groups. Column 4 shows the accumulated values of the data in column 3. The significance of column 4, for example, is that only 0.06 percent of the time is the discharge 2 c.f.s. or less, and 99.94 percent of the time the discharge is 2000 c.f.s. or less.

In connection with the computations for duration curves, it is of interest to note that for the East and West Forks of Deep River, and Muddy Creek, near High Point, N. C., curves were also computed directly from stage recorder charts as well as from the records of mean daily discharges. This procedure of determining the percentage of time that a particular discharge is equaled or exceeded gives the correct curve because instantaneous discharges are used instead of daily averages. A comparison of the curves as calculated by the above methods showed

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1/ This section was worked out and written by Joe W. Johnson, formerly Hydraulic Engineer, Soil Conservation Service.

Table 4.—Data for duration curve for Little Sugar Creek near Charlotte, N. C.*

Discharge	Number in the class	Proportion of time	Accumulative proportion	Q/Qm
C.f.s.		Percent	Percent	
0 - 1.9	3	.06		
2 - 3.9	94	1.72	.06	0.0437
4 - 6.9	483	8.82	1.77	.0873
7 - 9.9	625	11.41	10.59	.1528
10 - 19	1,705	31.13	22.00	.2183
20 - 39	1,418	25.89	53.13	.4366
40 - 69	537	9.81	79.02	.873
70 - 99	179	3.27	88.83	1.528
100-199	236	4.31	92.10	2.183
200-399	111	2.03	96.41	4.366
400-699	46	.84	98.44	8.73
700-999	20	.37	99.28	15.28
1000-1999	16	.29	99.65	21.83
2000-3999	4	.07	99.94	43.66
	5,477			

*Drainage area 41.4 sq. miles.

that the curve based on instantaneous discharges was practically coincident with the curve based on mean flows for lower rates of flow; it was only slightly below the mean curve for discharges in the vicinity of the average discharge, and was above the mean curve for the very highest discharges. This relative location of the two curves is to be expected because during low flows the instantaneous and the mean daily discharges are practically the same and the duration curves naturally will coincide. For the higher rates of discharge the highest values are obtained when instantaneous discharges are considered; hence, the curve based on instantaneous discharges will lie above the curve based on mean daily flows. A plot of the two curves on probability paper shows that, for all practical purposes, the curves coincide. Thus, because of the ease with which curves may be calculated from tabulations of mean daily discharges, the curves shown in figure 18 were computed by that method.

In order to compare the duration curves from several streams it has been found convenient to express the discharge as a ratio to the mean rather than as direct discharge. This procedure permits the curves to be compared directly by eliminating the size of the drainage area. Thus, for Little Sugar Creek the mean discharge for the period, 1925 - 1939, is 45.8 cubic feet per second. The ratio of the discharge at the lower limit of each discharge group to this mean discharge is given in column 5 of the accompanying table; that is, for example, 2/45.8 = 0.0437, 4/45.8 = 0.0873, etc.

A plot of the Little Sugar Creek data in columns 4 and 5 is shown in figure 18 along with the curves for several other Piedmont streams of varying sized drainage basins. Examination of these curves shows a remarkably close agreement for the upper 40 percent of the time; that is, for the higher rates of discharge the various curves are almost identical. For that part of the curves that represent the lower rates of flow, however, the curves radically depart from each other. This is to be expected because for low rates of flow, the drainage basin characteristics, such as soil types, vegetal cover, tillage methods, and stage of soil erosion, greatly influence the runoff, but during times when high precipitation rates cause high runoff rates the drainage basin characteristics affect the runoff very little.

Of interest in this respect is the comparison between the curves for the Uharie River and Muddy Creek with the curves for the East and West Forks of Deep River. The Uharie River and Muddy Creek watersheds lie in the slate belt of North Carolina where the depth of soil above bed rock is usually only from 0 to 3 feet; consequently, there is little space for ground water storage and runoff is rapid and flashy. With very little ground water storage available, the low-water flows are relatively small; hence the duration curves for these two streams plot relatively lower than the other streams shown in figure 18. On the other hand, the drainage basins of East and West Forks of Deep River lie in the granitic schist region of the Piedmont where weathering is relatively deep and a considerable depth of soil is available for ground-water storage. Thus, the low

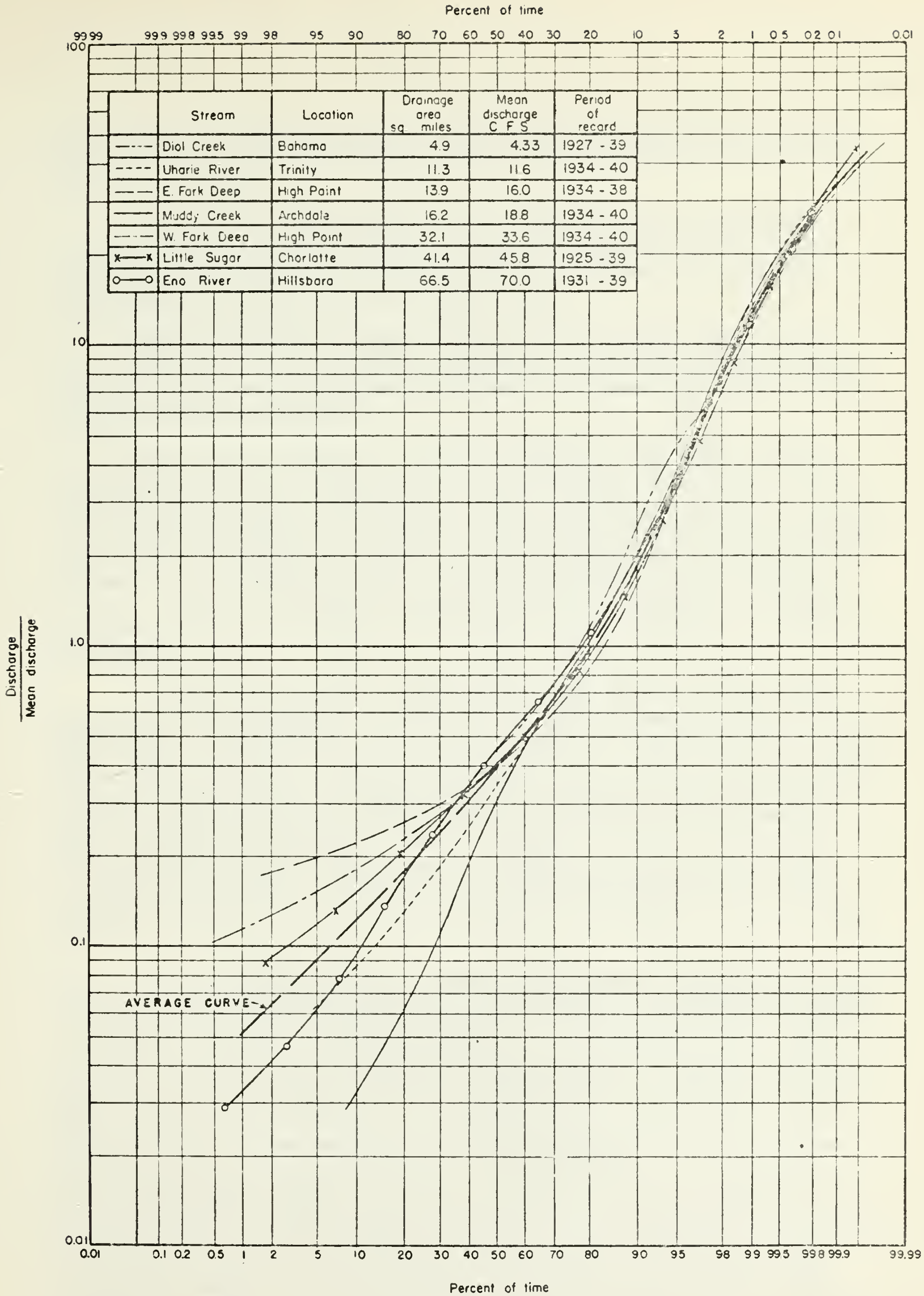

FIGURE 18

Flow duration curves for several small streams in the Piedmont region of North Carolina

water flows in these streams are relatively high and the portion of the duration curves representing these flows, lie higher than any of the other curves.

As will be shown below, it is fortunate that the duration curves for streams in the same region are identical for the higher rates of flow because it is during these flows that the greatest percentage of the bed load is transported.

The fact that the upper portion of the flow-duration curves are identical for streams in a particular region is very important when it is desired to estimate the load of streams that have no discharge records available.

Thus, for a stream in the North Carolina Piedmont region an average curve may be obtained and used in most calculations on bed-load transportation. This average curve (see figure 20) may not give an accurate estimate for low-water flows but little material is transported at such times and the errors in using the average curve will be small. To apply the average flow-duration curve to a particular stream, a value for the mean discharge must be assumed or estimated in order that the ordinates of the curve can be converted to actual discharge.

Previous hydrologic studies have shown that the mean annual discharge is, in general, related to the annual precipitation (11). That this relationship is not constant for individual years is evident from figure 19 which shows mean annual runoff in c.f.s. per sq. mile plotted against annual precipitation in inches for various streams in the Piedmont region of North Carolina as well as for streams in Iowa, Wisconsin, Washington, and the mountains of North Carolina. A close examination of this figure, however, reveals a reasonable explanation for the scattering of the individual points and with proper consideration and caution a usable relationship can be determined.

If 100 percent runoff occurred, the single straight line shown in the accompanying figure would describe the relation between rainfall and runoff. Because, however, a certain amount of rainfall is lost by evaporation and transpiration, the plotted points fall somewhat to the right of this 100 percent curve. Previous investigations (12) have shown that curves, depending on whether the streams are in mountainous regions or in valleys, can be drawn through the plotted points. The main effect of the location of a stream appears to be in the influence of length of growing season on evaporation and transpiration losses. Thus, as a guide in the construction of curves through the data plotted in figure 19 a plot was made of the annual water loss (represented by the horizontal distance from the 100 percent curve to the plotted point) against the average length of growing seasons for the county in which the watershed of the stream is located. Data on the length of growing season was obtained

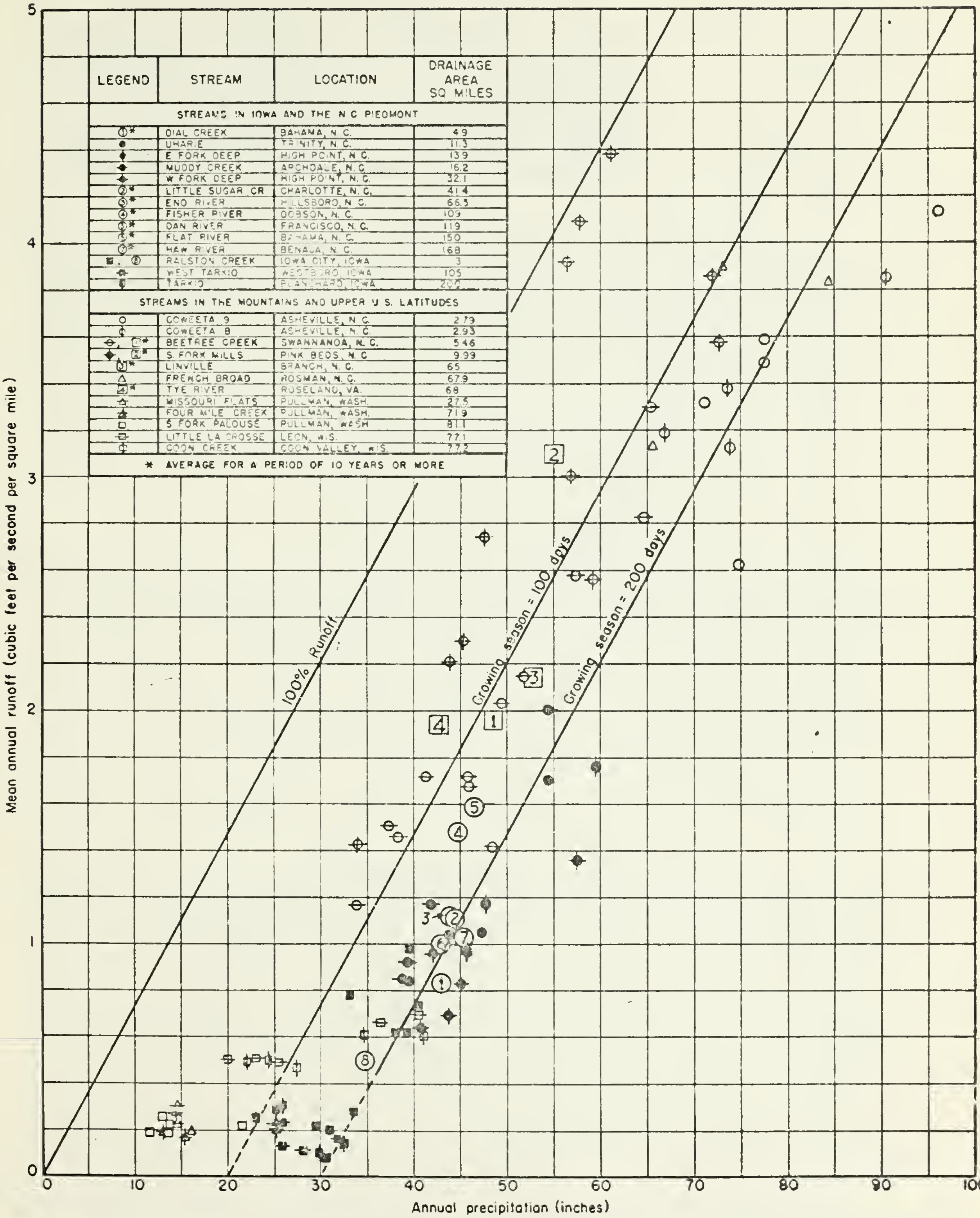

Figure 19.- Relation between mean annual runoff and annual precipitation for streams in various localities in the United States.

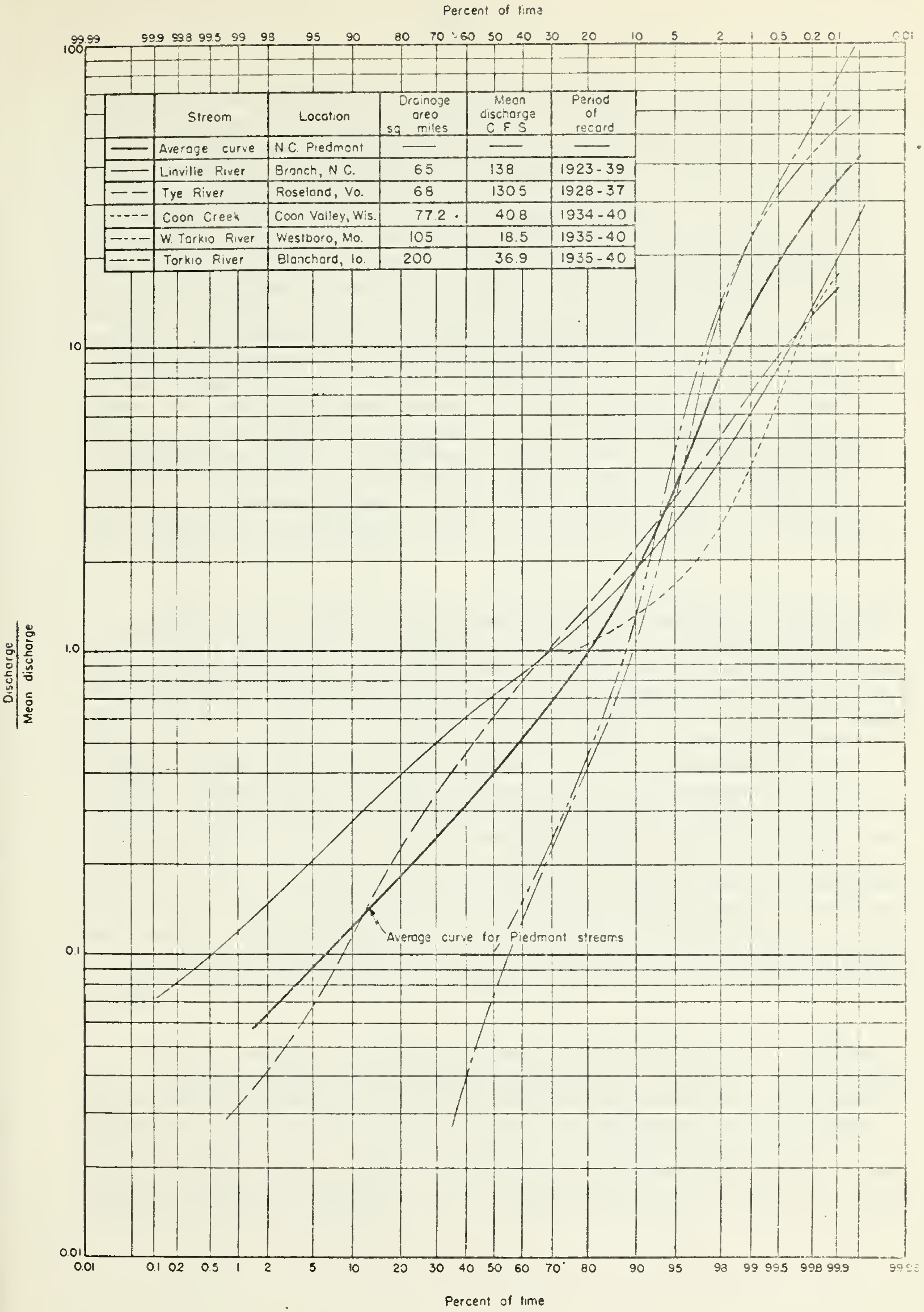

FIGURE 20

Flow duration curves for various streams compared with average curve for Piedmont streams.

from the 1941 Yearbook of Agriculture in which a climatic summary for each county and state in the United States is given (9). A straight line was then drawn through these plotted points. If it is now assumed that a certain base rainfall is required to supply evaporation and transpiration losses and that practically all precipitation greater than this base amount appears as runoff, curves can now be drawn in on the graph of rainfall vs. runoff (fig. 19). Thus, from the plot of water loss vs. length of growing season, a loss of 20 inches appeared to occur for a growing season of 100 days, and for a growing season of 200 days the loss was approximately 30 inches. By drawing lines through the base amounts of 20 and 30 inches of rainfall parallel to the 100 percent runoff line, the curves shown in figure 19 were determined. The line for a growing season of 100 days passes through the approximate center of the data for mountainous regions and regions in the northern latitudes of the United States. Furthermore, the curve for a growing season of 200 days passes through the data for streams in the Piedmont region and streams in Iowa. As a first approximation these curves were assumed to be straight lines, however, further study may show that they are curved lines.

The use of figure 18 is illustrated as follows: Assume that an estimate is desired of the annual volume of bed-load transported in a particular stream in the Piedmont region of North Carolina. If no discharge data are available on this stream, the average flow-duration curve from figure 20 is used. To convert the ordinates of this curve to actual discharge, the data shown in figure 19 are used. From the maps of average annual rainfall shown in the 1941 Yearbook of Agriculture (9), the annual rainfall for the watershed of the stream is determined. From the same source, the average length of growing season is also determined. Entering the scale of abscissa with the annual rainfall and proceeding upwards until the curve representing the average length of growing season is reached the mean annual runoff in cubic feet per second per square mile is determined. This value of mean discharge when multiplied by the area of the drainage basin in square miles gives the estimated mean discharge. This value may now be multiplied by each ordinate of the average duration curve to obtain the estimated flow duration curve for the stream under study.

As a matter of interest, flow-duration curves were computed for streams in other localities throughout the country. These curves are shown in figure 20 along with the average curve for the Piedmont streams. These various curves stress the importance of using data from streams in or as near as possible to the same locality as the streams under study. Snow conditions, underground losses, etc., may cause the duration curves to vary considerably for streams in various sections of the country.

It is, of course, acknowledged that the relations between rainfall and runoff as expressed by figure 19 are merely generalizations which bring out class likeness and obscure the individual characteristics of runoff from differences in the character and distribution of rainfall, and the effect of temperature, vegetal cover, topography, soil and subsoil on the disposal of rainfall. Although the method of arriving at an estimated flow-duration curve for a particular stream upon which all directly observed hydrologic data are missing, may not be fully compatible with more recent hydrologic research, it appears to be the most simple and direct approach in obtaining the necessary information to be used in estimating volumes of bedload transportation.

Application of the duration curve to Mountain Creek

The determination of the average duration curve for Mountain Creek by direct measurement was out of question because the record included only a few months of an especially dry year. The general type of the watershed is the same as for the North Carolina streams in figure 18 making a direct use of this curve possible. According to the procedure outlined previously, the average annual precipitation was found to be 52 inches for the watershed and the mean duration of the growing season 228 days (9, pp. 1103, 1107). With these two figures the mean annual runoff, 1.4 cfs/sq.mi., is obtained from figure 19. The total mean runoff, therefore, is

$$Q_m = 11.75 \times 1.4 = 16.45 \text{ cfs}$$

Using the relation between discharge and transportation as given in table 2, duration data are obtained, as shown in table 5.

Table 5.--Data for duration curve of flow and transportation in Mountain Creek

Bed-load Transportation per hour (Q_s)	Water depth (d)	Discharge (Q)	Q/Q_m	Proportion of Time
Lbs. under water	Ft.	c.f.s.		percent
10	0.18	3.06	0.186	20.0
30	0.24	4.51	0.274	32.0
100	0.33	7.20	0.438	52.0
300	0.47	11.7	0.711	71.0
1000	0.79	22.5	1.37	86.0
3000	1.43	47.4	2.88	93.8
5200	2.00	75.0	4.56	96.2

The duration curve of the transportation and of the water depth is plotted in figure 21. If the banks are assumed to be 2.0 ft. high (above the bed) this curve is only applicable up to this level. At greater depths the water spreads out over the flood plain where it becomes ineffective for transporting bed load. To describe this condition the curves would have to show a break at the point where $d = 2.0$ ft., making the increase above this point in both Q_s and d with respect to Q much slower. This increase has been fully neglected, resulting in a rather low estimate for the total, which is calculated in table 6 using the Simpson formula for integration.

Table 6.--Calculation of the average annual transportation of bed load by Mountain Creek

Proportion of time	Interval	Q_s Left Q_{s1}	Q_s Center Q_{s2}	Q_s Right Q_{s3}	$\frac{A(Q_{s1}+Q_{s2}+Q_{s3})}{6 \times 100}$
Percent	Percent	lbs/hr	lbs/hr	lbs/hr	lbs/hr
100-96	4	5200	5200	5200	208.
96-92	4	5200	3200	2200	134.7
92-88	4	2200	1630	1280	66.7
88-84	4	1280	1030	840	41.6
84-80	4	840	705	595	28.37
80-70	10	595	410	290	42.08
70-60	10	290	212	159	21.62
60-50	10	159	120	91	12.17
50-40	10	91	68	51	6.90
40-30	10	51	37	26	3.75
30-20	10	26	17	10	1.73
20-0	20				.38
Average rate of transportation $\overline{Q}_s$ - - - - - - - - -					568

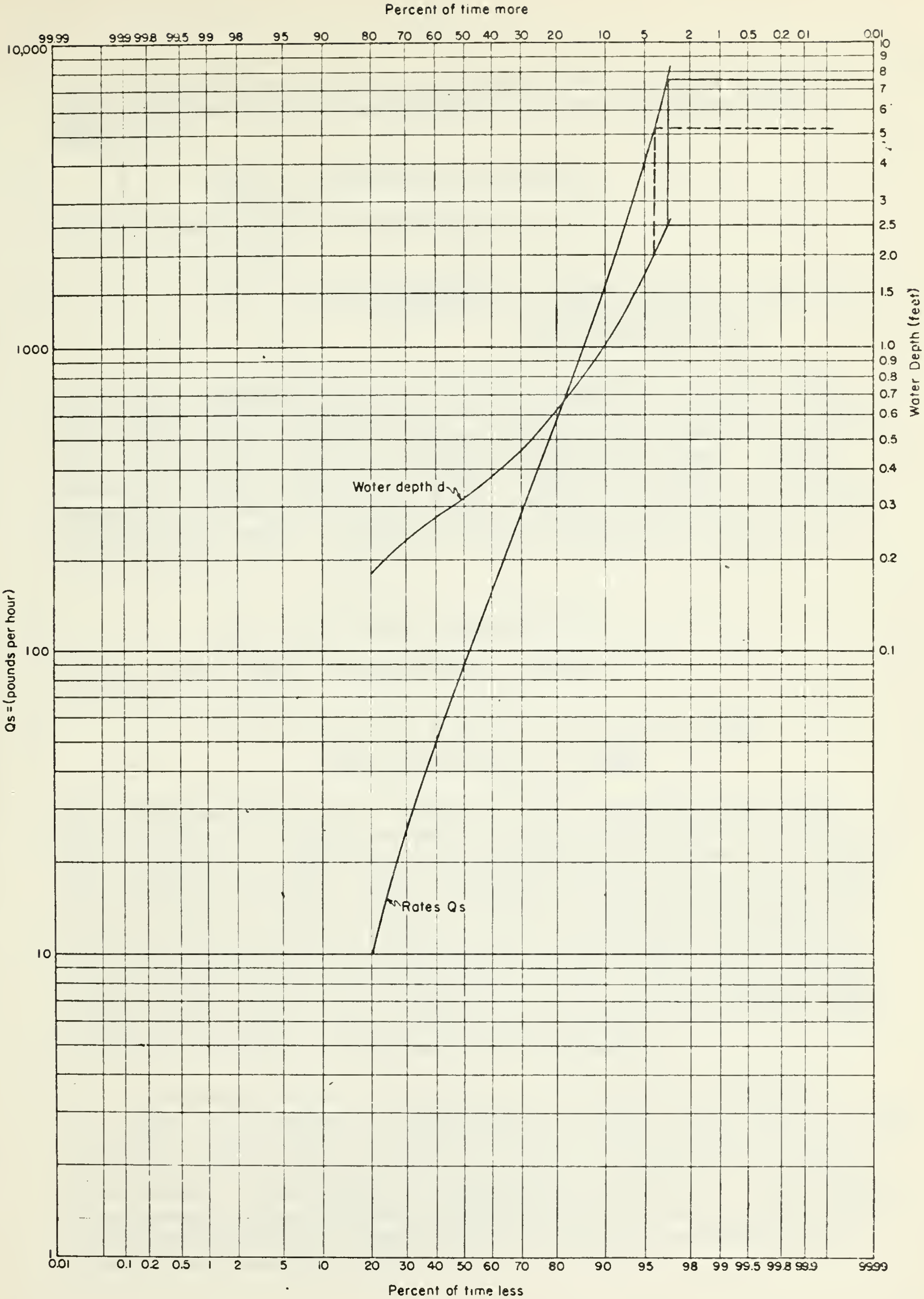

Figure 21 - Duration curves of transportation and water depth for Mountain Creek.

As the intervals are measured in percent, the sum of the integrals will represent the mean transportation, $\overline{Q}_s$, for an average year, which is

$$\overline{Q}_s = 568 \text{ lbs/hr. under water}$$

and the average annual transportation, A_s, is

$$A_s = \frac{568 \cdot 24 \cdot 365}{2000} = 2{,}490 \text{ tons/yr. under water}$$

or 3,980 tons/yr. dry
or 71,000 cu.ft./yr

by using a conversion factor of 70 lbs. under water for the cubic foot. This figure of 71,000 cu.ft. characterizes the cross section even better than the curve in figure 13 because it takes into account the frequency with which the different rates take place. On this basis it is again possible to compare different sections. A first comparison may be drawn between two sections that differ only in the height of the banks. A fully analogous determination for 2.5 foot banks will give (see figure 22).

$$\overline{Q}_s = 651 \text{ lbs/hr. under water}$$

and the annual total transportation, A_s, is

A_s = 81,400 cu.ft./yr., representing a 15% increase over 2.0 ft. banks.

This difference will become effective only during the high stages and therefore is significant only for long time averages. Calculating the transport for some other bank heights, figure 22 can be plotted, which gives the average annual transportation in terms of the height of the banks. It shows distinctly that an increase of transportation of about 10,000 cu.ft. per year, or 15%, can easily be achieved by dikes along the stream 6 inches high, a measure that will definitely pay for itself under certain conditions. The dashed curve shows the average annual duration of flooding as taken from the duration curve of the water depth. A deep channel will provide for both high transportation and short flooding as would be expected.

The twin diagrams of figures 14 - 17 show the influence of bed width, bed roughness, bank steepness, and bed slope on the ability to transport and on the tendency to flood. They are rather difficult to discuss without taking the frequency of the different discharges into account. By applying the duration curve they can be transformed into curves similar to those in figure 22 which give the total annual transportation and the average number of hours of overbank flow in terms of the same variables.

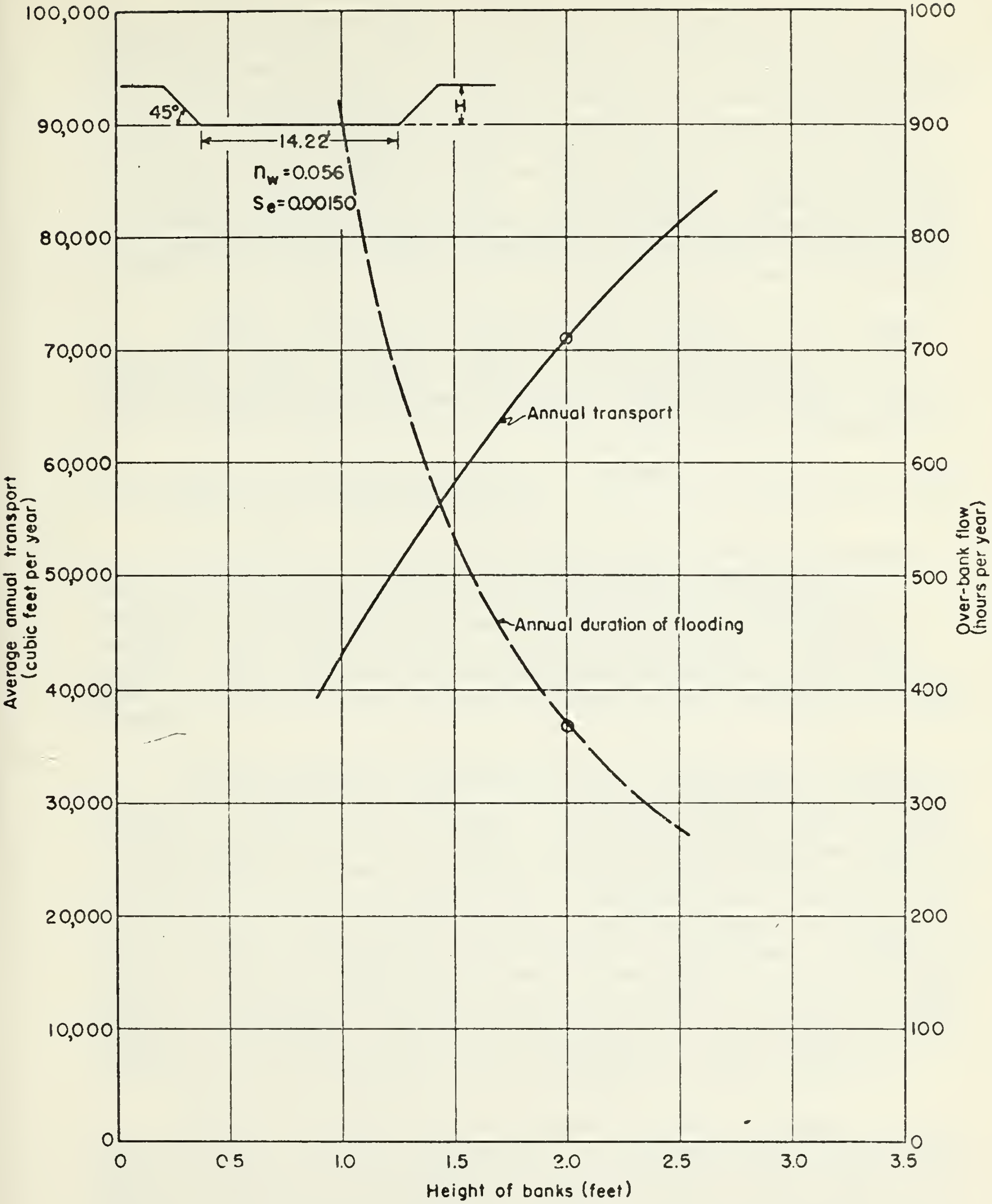

Figure 22 - Average annual transport of sand in Mountain Creek for various bank heights.

Figure 23 shows that increased bed width will lessen the danger of flooding by mere increase of the cross-sectional area. It has been shown in figure 14 that the same discharge will transport about even amounts of sand in the wide and in the narrow channel. The total annual transport, however, increases distinctly with the width of the channel because higher floods will remain within the banks. It may be mentioned here, that the contrary has been found in sections where overbank flow was unimportant and where the banks are smooth compared with the bed. The tremendous increase of transportation with decreasing roughness of the banks is shown much better in figure 24 than in figure 15 where the logarithmic scale tends to obscure this effect. The effect on flooding is not very pronounced.

Figure 25 gives the influence of the steepness of the banks. Again, as shown in figure 16, there is very little gain in having the banks steeper than 45°. However, flatter angles result in a rapid decrease of the sediment capacity. It is interesting, however, that in this latter case the capacity to hold water is increased.

Figure 26 again shows the rapid increase in capacity for water and sediment with increasing slope.

In general it may be emphasized that this method of describing the capacity of a stream by the mean annual transportation is much easier to understand and in many ways is more significant. However, as both river cross sections and the duration curve determine these curves, they very specifically describe a particular section. Any conclusions drawn from these curves are true only for this one stretch and may be much different or even reversed in another stretch.

For the practical application of the average annual transport, it is very important to keep in mind that this rate has two distinct meanings: First, it is the amount of sediment that a certain stretch of the stream is able to transport under the prevailing flow conditions without changes in the bed. Second, if there is any proof or reason to assume that no such changes of any consequence have occurred during recent years, it can be concluded that just this amount of sediment came down into this stretch from the watershed upstream. The difference between these two conceptions is easy to see when it is borne in mind that the capacity of the stretch can be changed by changing the local roughness conditions, etc., but this change could not possibly affect the rate of sediment produced by the watershed. This latter can be affected only by influences on the watershed above the stretch.

Another principle that can be used to good advantage in practical problems is that of continuity. In comparing the average annual transportation in two stretches of the same stream, the deposition in the bed between the two stretches is the capacity of the upper stretch plus the capacities of tributaries joining the stream between the two stretches minus capacity of the lower stretch. A negative deposition would mean scour.

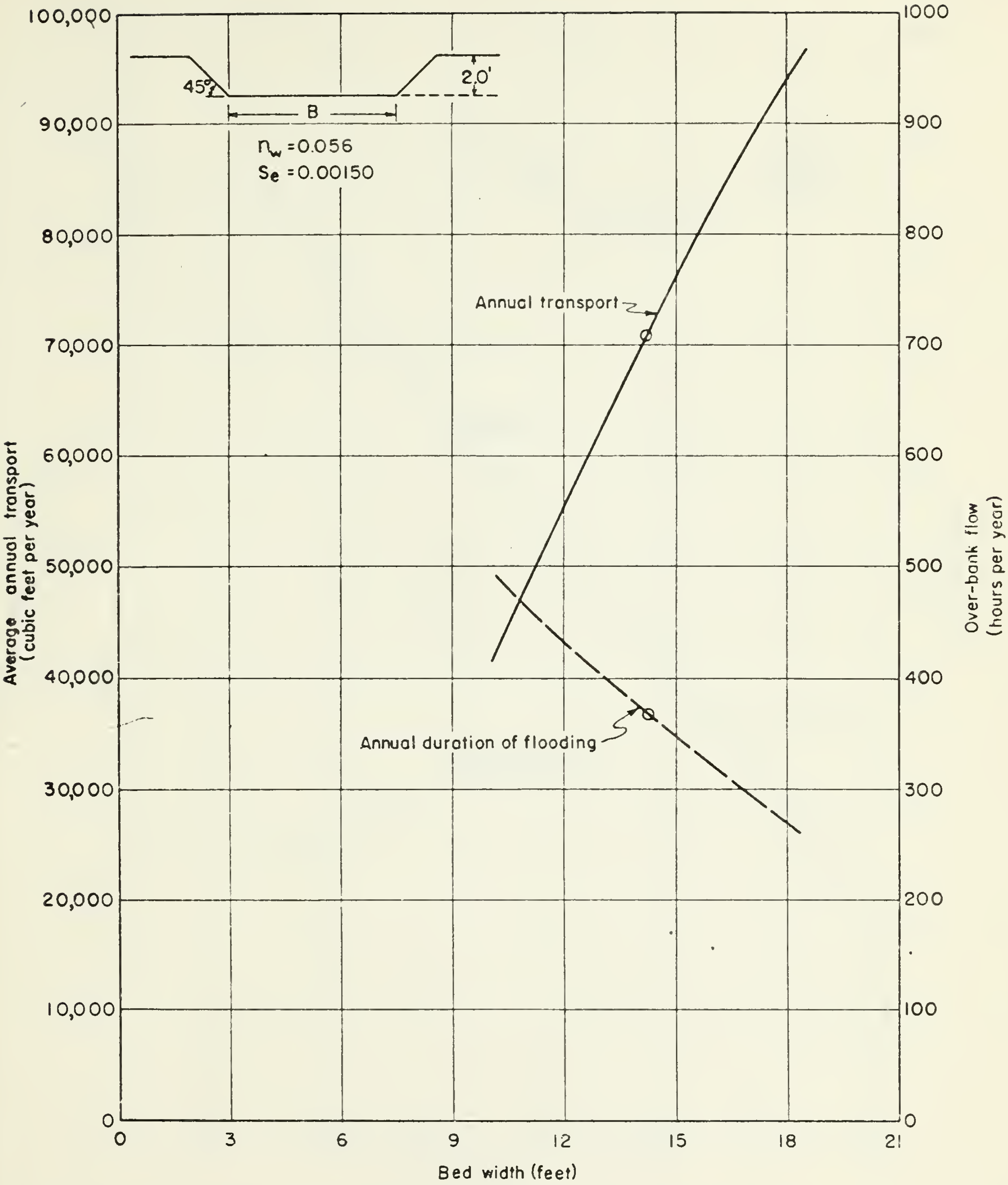

Figure 23 - Average annual transport of sand in Mountain Creek for various bed widths.

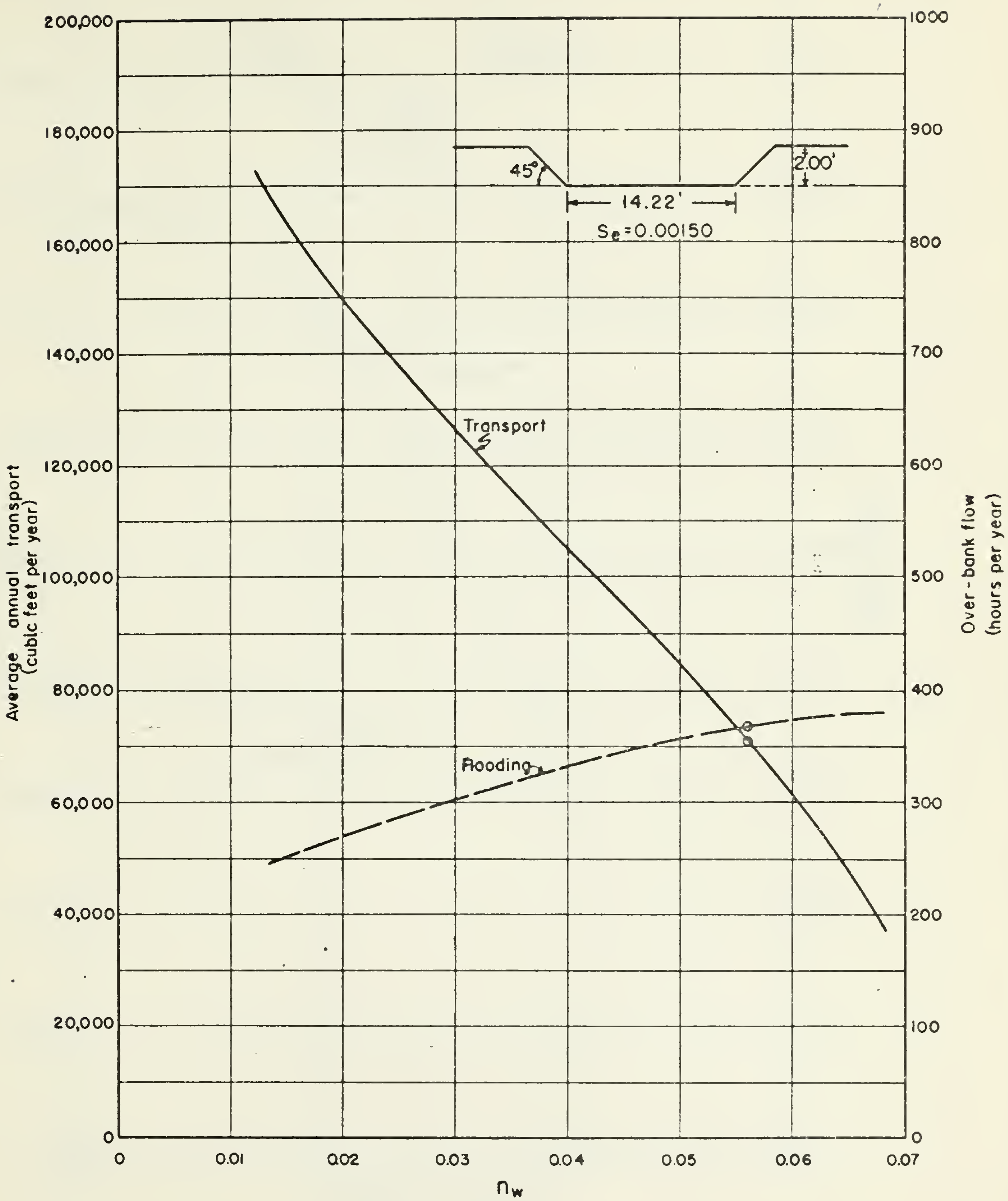

Figure 24 – Average annual transport of sand in Mountain Creek for various roughnesses of the banks.

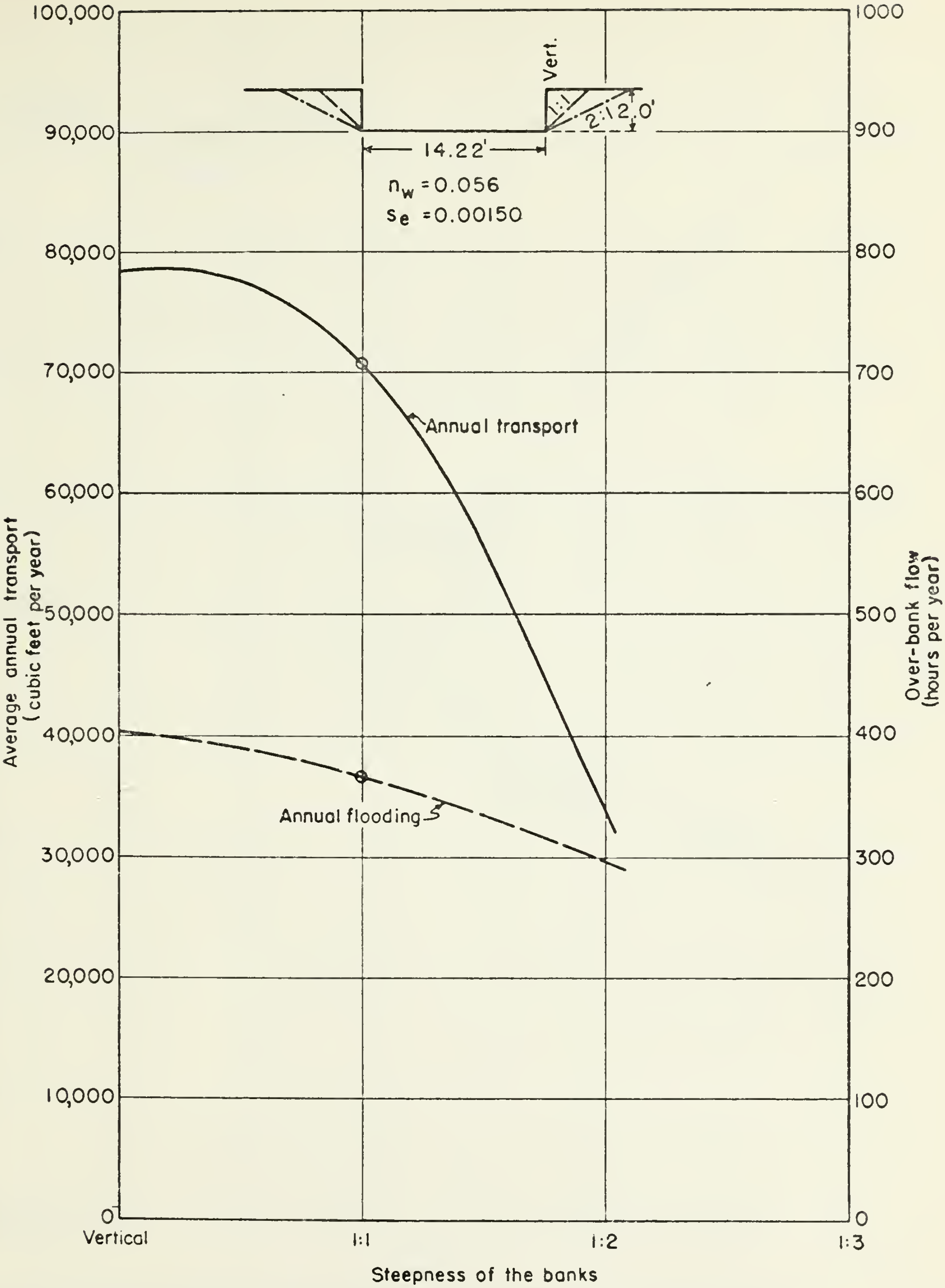

Figure 25 - Average annual transport of sand in Mountain Creek for various steepnesses of the banks.

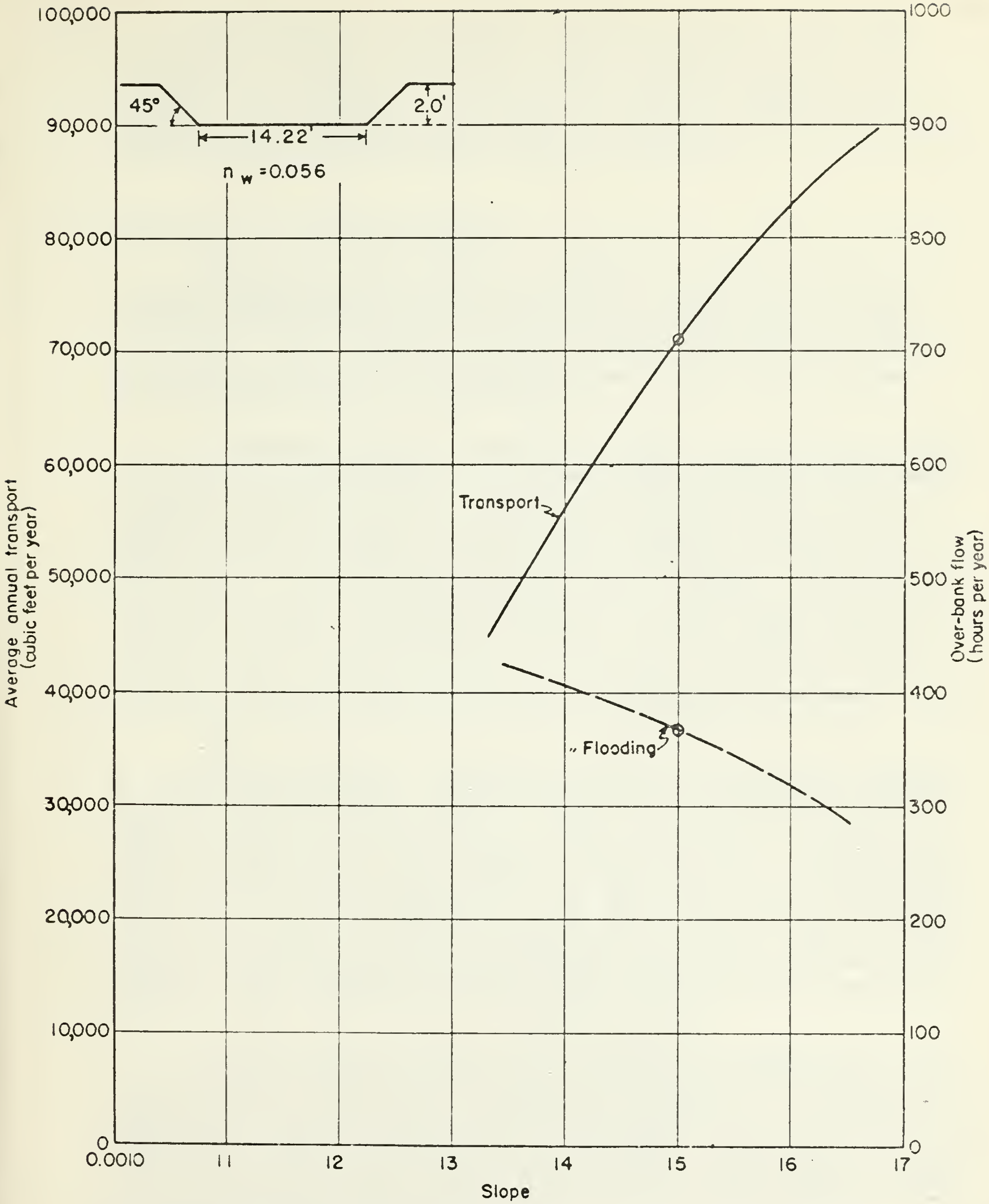

Figure 26- Average annual transport of sand in Mountain Creek for various slopes S_e.

In addition to these general principles for the application of sediment transport, it is well to keep in mind that every river is different from all other rivers. Satisfactory results from investigations of this kind depend mainly on the proper choice of the stretches used originally to describe the sediment output of the watershed in its present state and on the clever use of other "evidence" indicating the present trend of change in a stretch.

Application to practical problems

It has been shown that it is possible to predict the rate of bed-load transportation and water depth on a movable bed based only on certain measured hydraulic constants and on the analyses of the bed sediment. The principles involved can be applied in the solution of numerous practical soil conservation and flood control problems that heretofore have not been solvable.

An important problem, encountered usually in reconnaisance work, involves the determination of the annual load in different tributaries of a stream system in order to locate the most important sources of sediment. The procedure is to select representative reaches in the various tributaries with as uniform cross sections as possible and where normal flow for most discharges can be assumed. For each reach the curve for rate of transportation against discharge must be developed by using an "idealized section". The total annual transportation can then be estimated from a combination of this curve and a duration curve of the discharges. If a duration curve is not available for the stream under investigation, a curve based on other streams in the region and corrected for size of watershed may be used.

A second problem of interest arises when the supply of sediment is changed at the source. Where the clearing of land or the application of conservation measures will increase or decrease the load in a certain stream by a known amount, it may be necessary to determine the effect on the ability of the stream to drain the land. The questions are: Will the stream scour or fill and how much? Can excessive change of the stream bed be prevented and how? These questions are extremely important because of their effect on the bottom lands along the stream and are especially difficult to answer if both the load and the runoff change. These influences, however, can be separated and compared by means of a short study with an "idealized section".

A third problem exists in all rivers where a change of the load or the runoff or both have taken place in the past and the river is in the process of readjusting its profile and sections to the changed conditions. In this case, it is important to determine the final equilibrium and to improve conditions in the meantime. This problem involves the determination of the load in some part of the river where at least temporary equilibrium has been achieved and the routing of this load through other parts of the system.

A fourth problem is introduced when for any reason the old channel of a stream is to be replaced by a cutoff channel. As only one particular cross section will be stable, careful study of all conditions and all consequences connected with this new stream will be necessary to prevent undesirable developments after the rather expensive works are in operation.

These are only a few of the many problems that are continually developing in soil conservation and flood control work. Sometimes the problems are combined with a whole series of problems, influencing one another and thus making the simultaneous consideration of a whole river system necessary. Even then, most of them can be solved quantitatively only by frequent applications of a set of equations similar to that used to describe the flow in the "idealized section."

During the studies on Mountain Creek a phenomenon was observed that is not peculiar to this stream but may be found in many natural streams with a movable sand bed. At low water certain parts of the surface of the bed seemed to be composed entirely of large particles while on other parts no large particles were in evidence and the surface seemed to be made up of a well-graded combination of the smaller sizes. Naturally, there was the suggestion that these large particles might form a "protective layer" that would prevent local erosion and thereby decrease the rate of transportation. With this condition in mind the data were studied carefully but no discernible effects could be found. Special studies are now in progress at the Enoree River Laboratory, Greenville, S. C. to determine, if possible, if there are any conditions under which bed sediment composed of material from Mountain Creek will not be transported to capacity.

This study has opened a number of bed-load problems to analytical treatment but does not pretend to have completely solved the problem of bed-load transportation. An analytical method for the treatment of bed-load problems in general has been proposed and the necessary equations given together with certain rules about the description of sediment mixtures by representative grain diameters. The measurements in Mountain Creek have proved the applicability of this method to this stream and thereby to all other streams with values of $\Phi < 1$ 1/ and

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1/ According to the author's experience, the range of effective Φ - values in natural streams depends mostly on the grain size of the bed material and on the stage. Counting all Φ - values up to 10 covered by the measurements (see appendix II), in all streams with sediment of 1 mm. or coarser representative grain size the bed-load movement can be calculated, excluding the part of the bed material moving in suspension. For sediments with a representative diameter down to 1/2 mm. bed-load movement at low and intermediate stage will be covered, excluding suspension. For still finer sediment the movement as bed load seems insignificant compared with suspended load for cases where the total load is comparatively small.

sediment mixtures with the same or smaller spread of grain sizes. The spread of grain sizes is given by the slope of the cumulative distribution curve as shown in figure 9 for Mountain Creek. The logarithmic-probability plot is especially appropriate because in this type of chart most river sediments plot almost as straight lines, very clearly defining this slope.

There is no evidence indicating that the applicability of the method is confined to the limits derived from the measurements in Mountain Creek but it has not been verified beyond them. The extension of the limits of applicability can be achieved only by systematic continuation of similar measurements in streams with different flow and sediment characteristics combined with the necessary flume studies. However, the range covered by the measurements in Mountain Creek includes a great number of streams, especially small streams and larger ones with coarser sediment.

Resume and Conclusions

1. An apparatus for measuring the rate of bed-load transportation in natural streams has been developed. The capacity of the unit is 5000 lbs. (under water) per hour.

2. This apparatus has been used to measure the rate of transportation in small streams in the South Carolina Piedmont. A full set of hydraulic measurements was made simultaneously. The rates covered a range of from 20 to 3300 lbs/hr. under water, representing concentrations of 53 to 445 ppm. The amount of bed material in suspension was small compared with this and could, therefore, be neglected. Finer material in suspension was not measured because this part of the load cannot be related to the discharge.

3. The various measurements fully prove the applicability to this stream of the equations for the transportation of bed load and for the friction of a movable bed as derived from laboratory studies.

4. It has been shown that the duration curve of the discharge for an average year can be used to advantage in the determination of the average annual load. In cases where such a curve is not available for the stream under consideration, it can be derived from the duration curve for another stream with similar watershed conditions. Based on the duration curve the average annual bed load for Mountain Creek is estimated to be 71,000 cu.ft. or 1.63 acre ft. or 9.5 cu.ft/acre.

5. A comparatively quick analytical method is developed for calculating the carrying capacity of a natural stream section for both water and sediment using the "idealized section".

6. A study of the idealized section brings out very clearly the effects, when considered separately, of variations in such influences on the capacity of a stream for both water and bed load as width of the bed, roughness of the banks, steepness of the banks, and general slope.

7. The influence of the roughness of the banks on the transporting capacity of a stream is so pronounced that it suggests an unexpected and effective means of regulating that capacity, especially in smaller sized streams.

8. The applicability of the analytical method to soil conservation and flood control problems is manifold. Similar studies are necessary to prove the applicability of the method to larger streams and finer sediment. (See Appendix II.)

APPENDIX I.

DETERMINATION OF THE REPRESENTATIVE GRAIN DIAMETER

The entire evaluation of the measurements in Mountain Creek is based on the choice of the representative grain diameter of the bed mixture. As this choice was based on a purely empirical rule of the most primitive type and because the conclusions from these measurements seemed to be of basic importance, it was decided to check this diameter by means of a flume study.

This investigation was made by Mr. Alvin G. Anderson in close collaboration with the writer as part of a special study to determine whether every sediment mixture can be described in a bed-load formula by a constant, representative grain diameter. The equipment and the rather unique and fast-working method used in these experiments will be described in another paper. For the present only the results of one series of experiments using original Mountain Creek sand for the bed will be given. In these experiments the discharge and the rate of transportation were measured directly. Frequent sets of cross sections gave location and slope of bed and water surface. The roughness of the side walls had been previously determined.

The correctness of the assumed grain diameter is tested by assuming the same value, D_{35} = 0.068 cm. = 0.00223', that was used for the Mountain Creek data, introducing the measured side wall friction and plotting the results of the experiments in a normal Ψ - Φ graph. If the points fall on the curve, the assumed D is correct. Using the formulas

$$\Psi = \frac{\rho_s - \rho_f}{\rho_f} \frac{D}{SR} = \frac{1.65 \cdot 0.00223}{SR} = \frac{0.00368}{SR}$$

and

$$\Phi = \frac{q_s}{Fg(\rho_s - \rho_f)} \sqrt{\frac{\rho_f}{\rho_s - \rho_f}} \frac{1}{g^{1/2} D^{3/2}} = \frac{q_s}{234.5} = 0.00426\ q_s$$

the following values for Ψ, Φ and n_b are obtained:

Table 7.--Values of Ψ, Φ, and n_b from special flume experiment

Run	Q	d	V	S_w	S_e	R_w	Re_w	f_w	R_b	n_b	Ψ	q_s	Φ
	(cfs)	(ft)	(ft/s)	0.00	0.00	(ft)			(ft)			$\frac{lbs}{ft \times s}$	
1	0.235	.188	1.250	1565	1605	.085	9580	.0225	.156	.0138	14.70	2.44	.0104
2	0.43	.284	1.514	1451	1521	.1176	16000	.0201	.218	.0139	11.10	7.21	.0303
3	0.65	.391	1.662	1347	1434	.145	21700	.0194	.278	.0144	9.23	14.72	.0628
4	1.04	.585	1.778	1540	1571	.151	34000	.0193	.408	.0182	5.74	42.64	.1818
5	0.53	.339	1.563	1438	1504	.125	17600	.0198	.255	.0149	9.59	13.54	.0577
6	0.345	.243	1.420	1519	1572	.104	13300	.0208	.192	.0138	12.19	5.46	.0224

The values Ψ and n_b are plotted in figure 27 against Φ and compared with the curves given in figure 11. The deviations are small, proving that the value used for the diameter is correct within the allowable limits of error.

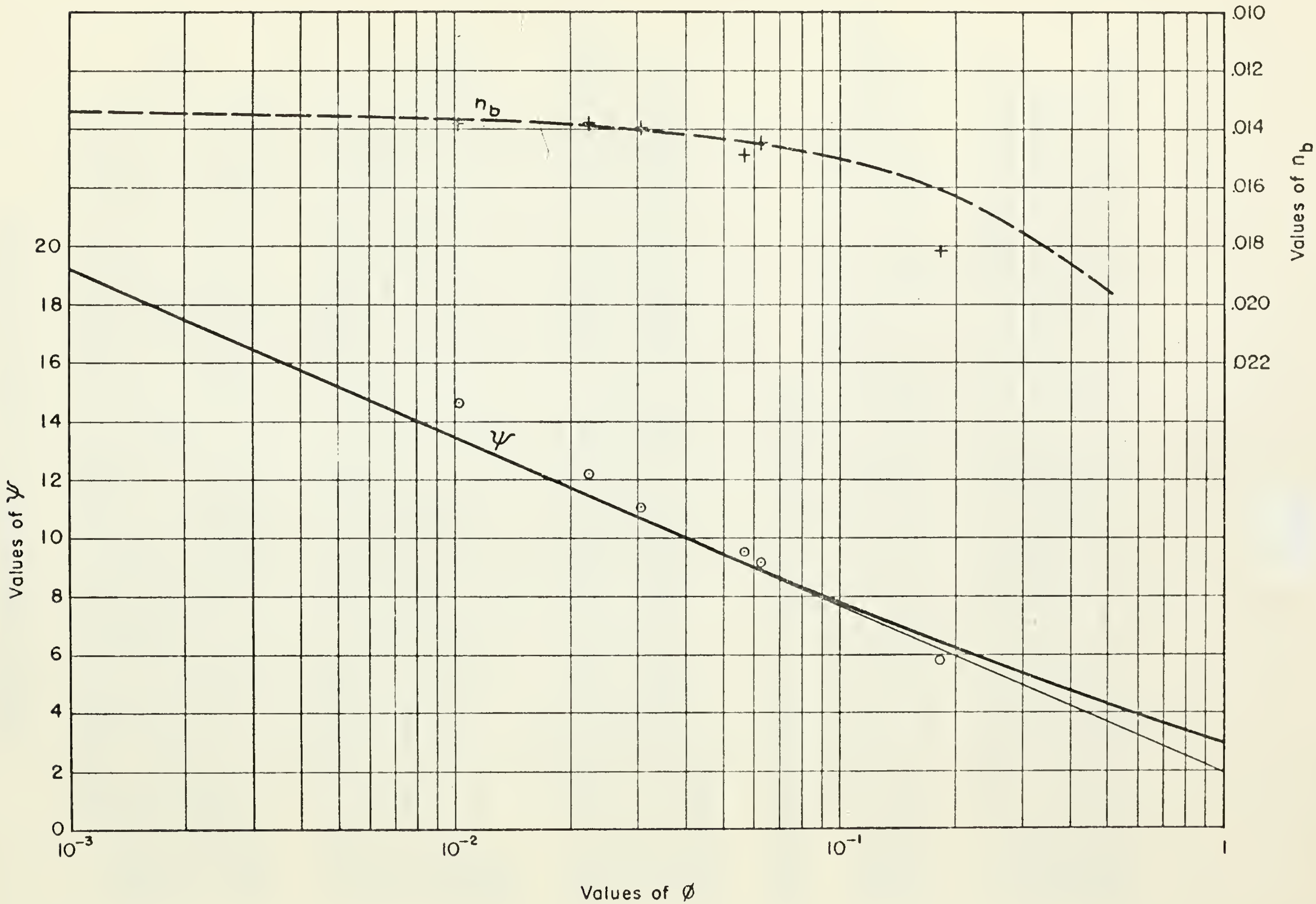

Figure 27 – Variation of n_b and ψ_b with the rate of transportation for the flume experiments using Mountain Creek sand in the bed.

APPENDIX II.

BED-LOAD MEASUREMENTS IN WEST GOOSE CREEK

West Goose Creek is a tributary of Tobitubby Creek in the Tallahatchie River basin approximately four miles west of Oxford, Mississippi. This stream, which along with its drainage basin has been extensively described in U. S. Dept. of Agri. Tech. Bull. No. 695 (6), is characterized by a channel plug near its mouth. As a result of the plug deposition has progressed upstream by backfilling. The stream above the plug is characterized essentially by an excessive supply of fine sand from innumerable gullies in the drainage basin.

In the reach of West Goose Creek immediately above the bridge on the old Batesville road (see figure 15, reference (6)), the cross section of the stream is very similar to that of Mountain Creek. The average width of the sand bed is 12.87 ft., and the mean diameter of the bed material is approximately 0.33 mm. as averaged from several samples taken throughout the reach. The stream banks have an average slope of 1:1, but appear relatively smoother than the banks of Mountain Creek because a large portion of the side slope is covered with fine sand.

Preliminary calculations show that with these channel characteristics and bed material, values of Φ will be from 5 to 10 times greater in West Goose Creek than in Mountain Creek for the same rates of transportation. Actual field measurements of data on bed-load transportation and bed roughness, therefore, seemed to promise a desirable extension of the range of conditions encountered in the Mountain Creek measurements.

Experimental apparatus and measuring reach

The same equipment as used in the Mountain Creek studies, figure 3, was used on West Goose Creek (see figure 28). For convenience and portability, however, the complete pumping unit and weighing tanks were mounted on the chassis of a 2-ton truck. A single hopper 10 ft. long and 2 ft. wide in the direction of flow was used.

Suspended load samples were taken from a submerged board walk fastened directly to the hopper, figure 28. This walkway served the additional purpose of preventing the scour of the fine bed material immediately downstream from the hopper, and proved superior to the bridge type structure used in Mountain Creek, figure 1, because no obstructions were located in the channel against which floating trash could accumulate.

Figure 28.--Measuring device on West Goose Creek.

The hopper was located approximately 50 feet upstream from the bridge on the old Batesville road. The measuring reach extended 550 ft upstream and included 12 ranges spaced at 50 ft intervals. Automatic stage recorders were provided at the two ends of the reach. Cross sections, slope measurements, and other observations were made in the same manner as in the Mountain Creek study. The results of the various measurements for two consecutive floods on March 23 and 24, 1942, are shown in table 8. Due to operational difficulties no measurements of rates of bed-load transportation were made during the rising stages of these floods; however, the fact that measurements were made only during the falling stages does not diminish their value, because no differences in the characteristics of bed-load movement between rising and falling stages is expected with a well-sorted sand such as that in West Goose Creek, figure 29.

Discussion of results

In summarizing the various data from West Goose Creek shown in table 8, the same formulas and methods were used as in analyzing the Mountain Creek data, except for the following slight modifications:

(1) The sand bed was so nearly horizontal across the stream at most cross sections that it was assumed as horizontal in calculating the elements of the sections.

(2) The slope was measured individually for each measuring period.

(3) After preliminary calculations the value of the Manning roughness coefficient (n_w) for the banks was found to be between 0.020 and 0.030. A value of 0.025 gives results for the transportation and bed friction that distinctly follow the trend of the Mountain Creek measurements.

(4) Because the value of the ratio D/δ, of the grain diameter divided by the thickness of the laminar sub-layer, lies between 0.85 and 1.4, the friction along the bed must be computed by a formula that covers the transition region between flows along smooth and rough walls. For this computation von Kármán's formula is used:

$$\frac{V}{\sqrt{\tau_o/\rho_f}} = \Gamma\left\{y + 5.75 \log_{10}\left(\frac{R_b}{D_{65}}\right)\right\}$$

wherein

Date	Time	Duration of run, minutes	Rate of transportation, * Q_s	0.030				
				$\sqrt{\frac{\tau_0}{\rho}}$ ft/sec.	$\frac{V}{\sqrt{\frac{\tau_0}{\rho}}}$	$\frac{D_{65}}{\delta}$	y	$\sqrt{\quad}$
(1)	(2)	(3)	(4)	(35)	(36)	(37)	(38)	(39)
3-26-'42	6:20	20	375	.172	25.7	1.23	7.40	1.18
"	6:45	30	298	.168	22.0	1.20	7.41	1.02
"	7:15	30	194	.158	20.4	1.14	7.43	.956
"	7:45	30	100	.152	18.4	1.09	7.44	.874
"	8:15	30	310	.145	16.9	1.04	7.44	.815
"	8:45	30	102	.137	16.0	0.98	7.44	.782
"	9:15	30	86	.133	15.6	0.95	7.44	.769
"	9:45	30	74	.130	15.4	0.93	7.44	.765
"	10:15	30	40	.128	15.1	0.92	7.44	.754
"	10:45	30	38	.129	15.0	0.93	7.44	.751
"	11:15	30	40	.125	15.0	0.90	7.44	.755
"	11:45	30	38	.123	14.6	0.88	7.44	.736
"	12:15	30	26	.118	14.7	0.85	7.43	.754
"	12:45	30	26	.117	14.1	0.84	7.43	.722
"	13:15	30	160	.117	14.1	0.84	7.43	.722
"	13:45	30	160	.116	14.2	0.83	7.43	.728
-27-'42	6:45	90	873	.154	18.2	1.10	7.43	.860
"	8:45	150	360	.125	16.6	0.90	7.44	.812
"	13:00	360	333	.111	12.2	0.80	7.41	.636

TABLE 8.

SUMMARY OF DATA ON BED-LOAD TRANSPORTATION AND BED ROUGHNESS

OBSERVED IN WEST GOOSE CREEK, OXFORD, MISS.

Date (1)	Time (2)	Duration of run, minutes (3)	Rate of transportation,* Q_s pounds under water per hour (4)	Stage feet: Range 11 (5)	Stage feet: Range O (6)	Stage feet: Average (7)	Slope, S (8)	Mean velocity, feet per second (9)	Area, A square feet (10)	Wetted perimeter of banks, P_w-feet (11)	$\phi = \frac{Q_s}{300}$ (12)
3-26-'42	6:20	20	3750	11.30	9.82	10.56	.00269	4.43	8.36	1.75	12.50
"	6:45	30	2980	11.18	9.67	10.42	.00275	3.70	6.41	1.36	9.90
"	7:15	30	1940	11.09	9.57	10.33	.00276	3.24	5.17	1.10	6.47
"	7:45	30	1000	11.05	9.48	10.26	.00285	2.80	4.22	0.905	3.33
"	8:15	30	3100	11.01	9.41	10.21	.00291	2.45	3.55	0.764	10.30
"	8:45	30	1020	10.99	9.36	10.17	.00296	2.20	3.01	0.650	3.40
"	9:15	30	860	10.98	9.32	10.15	.00302	2.08	2.75	0.594	2.86
"	9:45	30	740	10.97	9.31	10.14	.00302	2.00	2.61	0.565	2.46
"	10:15	30	400	10.97	9.29	10.13	.00306	1.94	2.48	0.538	1.33
"	10:45	30	380	10.98	9.29	10.13	.00307	1.94	2.48	0.538	1.27
"	11:15	30	400	10.96	9.28	10.12	.00306	1.88	2.35	0.510	1.33
"	11:45	30	380	10.96	9.26	10.11	.00309	1.80	2.22	0.480	1.27
"	12:15	30	260	10.95	9.25	10.10	.00309	1.74	2.08	0.542	.868
"	12:45	30	260	10.95	9.24	10.09	.00311	1.65	1.95	0.424	.868
"	13:15	30	160	10.95	9.24	10.09	.00311	1.65	1.95	0.424	.533
"	13:45	30	160	10.94	9.24	10.09	.00309	1.65	1.95	0.424	.533
3-27-'42	6:45	90	873	11.02	9.50	10.26	.00276	2.80	4.50	0.962	2.91
"	8:45	150	360	10.97	9.33	10.15	.00248	2.08	3.01	0.650	1.20
"	13:00	360	333	10.92	9.19	10.05	.00315	1.36	1.69	0.368	1.11

$n_w = 0.020$

Date (1)	Time (2)	R_w feet (13)	R_b feet (14)	ψ_b (15)	$\frac{R_b}{D_{65}}$ (16)	$\sqrt{\frac{\tau_o}{\rho}}$ ft./sec. (17)	$\frac{V}{\sqrt{\frac{\tau_o}{\rho}}}$ (18)	$\frac{D}{\delta}$ (19)	y (20)	Γ (21)
3-26-'42	6:20	1.230	.482	1.04	446	.204	21.7	1.46	7.32	.963
"	6:45	.928	.400	1.23	370	.188	19.7	1.35	7.37	.888
"	7:15	.756	.337	1.45	312	.173	18.7	1.24	7.40	.861
"	7:45	.594	.286	1.65	265	.162	17.3	1.16	7.42	.811
"	8:15	.479	.247	1.88	229	.152	16.1	1.09	7.43	.767
"	8:45	.400	.214	2.13	198	.143	15.4	1.02	7.44	.745
"	9:15	.363	.197	2.27	183	.138	15.0	0.99	7.44	.733
"	9:45	.343	.188	2.38	174	.135	14.8	0.97	7.44	.729
"	10:15	.324	.179	2.47	166	.133	14.6	0.95	7.44	.723
"	10:45	.324	.179	2.46	166	.133	14.6	0.95	7.44	.723
"	11:15	.309	.170	2.60	157	.130	14.5	0.93	7.44	.722
"	11:45	.288	.162	2.70	150	.127	14.2	0.91	7.43	.712
"	12:15	.273	.150	2.92	139	.122	14.3	0.87	7.43	.724
"	12:45	.251	.143	3.04	133	.120	13.7	0.86	7.43	.698
"	13:15	.251	.143	3.04	133	.120	13.7	0.86	7.43	.698
"	13:45	.253	.143	3.06	133	.119	13.8	0.85	7.42	.706
3-27-'42	6:45	.606	.305	1.60	282	.165	17.0	1.18	7.42	.791
"	8:45	.422	.213	2.56	197	.130	15.9	0.93	7.44	.770
"	13:00	.186	.126	3.40	117	.113	12.0	0.81	7.41	.622

$n_w = 0.025$

Date (1)	Time (2)	R_w feet (22)	R_b feet (23)	ψ_b (24)	$\frac{R_b}{D_{65}}$ (25)	$\sqrt{\frac{\tau_o}{\rho}}$ ft./sec. (26)	$\frac{V}{\sqrt{\frac{\tau_o}{\rho}}}$ (27)	$\frac{D_{65}}{\delta}$ (28)	y (29)	Γ (30)
3-26-'42	6:20	1.722	.415	1.21	384	.190	23.4	1.36	7.37	1.05
"	6:45	1.298	.361	1.36	334	.179	20.7	1.28	7.40	.945
"	7:15	1.059	.311	1.57	2.88	.167	19.4	1.19	7.41	.901
"	7:45	.830	.270	1.75	250	.158	17.8	1.13	7.42	.839
"	8:15	.669	.236	1.97	219	.149	16.4	1.07	7.43	.785
"	8:45	.560	.206	2.22	191	.140	15.7	1.01	7.44	.763
"	9:15	.508	.190	2.35	176	.136	15.3	0.98	7.44	.752
"	9:45	.480	.182	2.46	169	.133	15.0	0.96	7.44	.740
"	10:15	.453	.174	2.54	161	.131	14.8	0.94	7.44	.735
"	10:45	.452	.174	2.54	161	.131	14.8	0.94	7.44	.735
"	11:15	.432	.165	2.67	153	.128	14.7	0.91	7.43	.736
"	11:45	.403	.157	2.78	145	.125	14.4	0.90	7.43	.725
"	12:15	.382	.146	2.99	135	.121	14.4	0.86	7.42	.732
"	12:45	.351	.140	3.10	130	.119	13.9	0.85	7.42	.710
"	13:15	.351	.140	3.10	130	.119	13.9	0.85	7.42	.710
"	13:45	.354	.140	3.12	130	.118	14.0	0.85	7.42	.715
3-27-'42	6:45	.847	.287	1.70	265	.160	17.5	1.15	7.43	.820
"	8:45	.590	.204	2.67	188	.128	16.3	0.91	7.44	.795
"	13:00	.260	.124	3.46	115	.112	12.1	0.87	7.43	.629

$n_w = 0.030$

Date (1)	Time (2)	R_w feet (31)	R_b feet (32)	ψ_b (33)	$\frac{R_b}{D_{65}}$ (34)	$\sqrt{\frac{\tau_o}{\rho}}$ ft./sec. (35)	$\frac{V}{\sqrt{\frac{\tau_o}{\rho}}}$ (36)	$\frac{D_{65}}{\delta}$ (37)	y (38)	Γ (39)
3-26-'42	6:20	2.27	.341	1.47	316	.172	25.7	1.23	7.40	1.18
"	6:45	1.70	.318	1.54	294	.168	22.0	1.20	7.41	1.02
"	7:15	1.39	.283	1.73	262	.158	20.4	1.14	7.43	.956
"	7:45	1.09	.251	1.89	232	.152	18.4	1.09	7.44	.874
"	8:15	0.87	.224	2.07	207	.145	16.9	1.04	7.44	.815
"	8:45	0.74	.197	2.32	182	.137	16.0	0.98	7.44	.782
"	9:15	0.67	.183	2.44	170	.133	15.6	0.95	7.44	.769
"	9:45	0.63	.175	2.56	162	.130	15.4	0.93	7.44	.765
"	10:15	0.60	.168	2.62	156	.128	15.1	0.92	7.44	.754
"	10:45	0.59	.168	2.62	156	.129	15.0	0.93	7.44	.751
"	11:15	0.57	.160	2.76	148	.125	15.0	0.90	7.44	.755
"	11:45	0.53	.153	2.86	142	.123	14.6	0.88	7.44	.736
"	12:15	0.51	.140	3.12	130	.118	14.7	0.85	7.43	.754
"	12:45	0.46	.136	3.19	126	.117	14.1	0.84	7.43	.722
"	13:15	0.46	.136	3.19	126	.117	14.1	0.84	7.43	.722
"	13:45	0.46	.136	3.22	126	.116	14.2	0.83	7.43	.728
3-27-'42	6:45	1.117	.267	1.83	247	.154	18.2	1.10	7.43	.860
"	8:45	0.776	.195	2.80	181	.125	16.6	0.90	7.44	.812
"	13:00	0.343	.121	3.54	112	.111	12.2	0.80	7.41	.636

* Uncorrected for tank efficiency

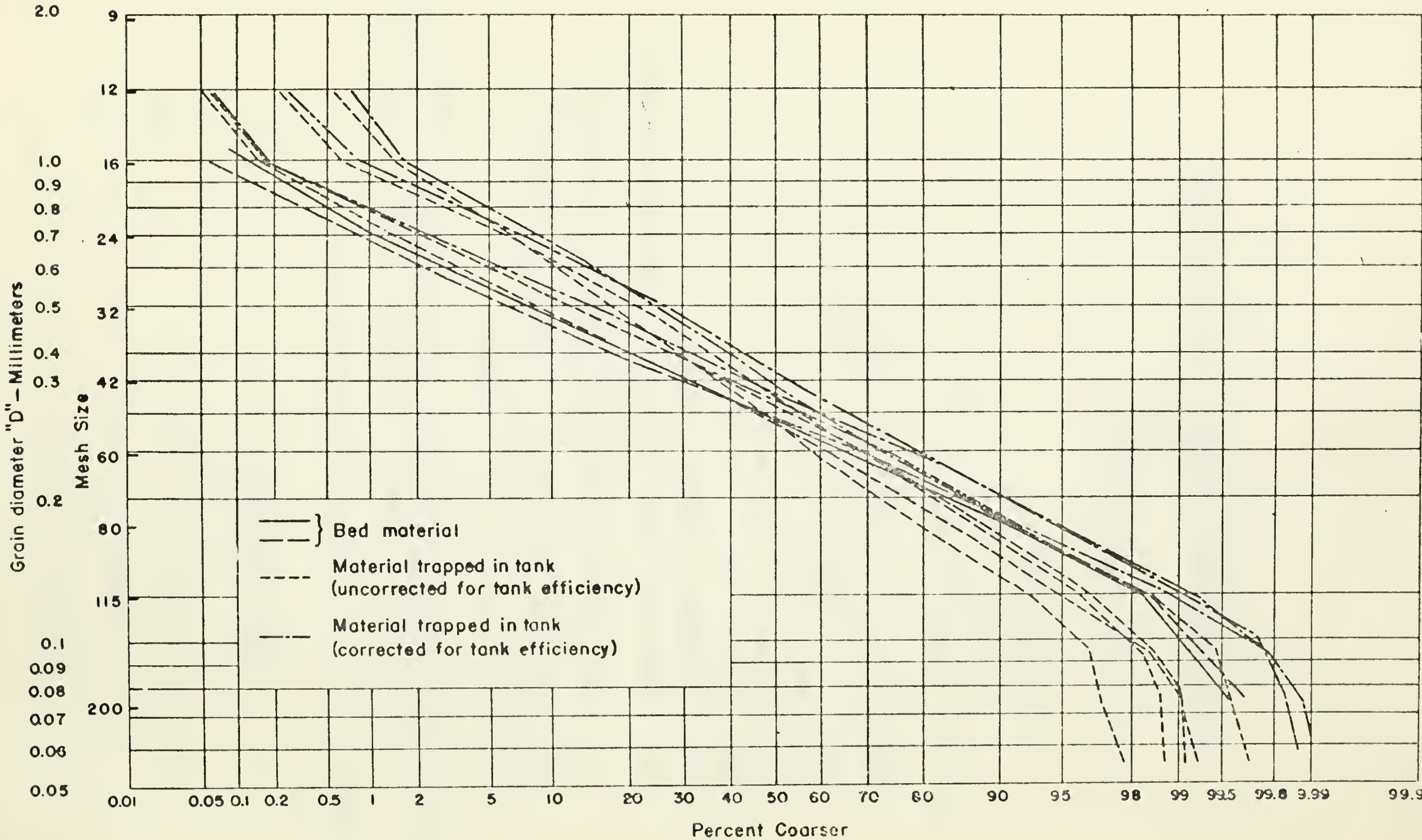

Figure29-Mechanical composition of sediment in West Goose Creek

$\tau_o = S_e R_b g \rho_f$ is the shearing stress of the stream along the bed

Γ is a correction factor for V and a function of Φ

while y is a function of D_{65}/δ (5)

$$\frac{D_{65}}{\delta} = \frac{D_{65}\sqrt{\tau_o/\rho}}{11.6\ \nu}$$

wherein ν is the kinematic viscosity of the water.

This function as given in figure 30 is derived for the case of open flow in a trapezoidal cross section according to Keulegan (8). The following constants have been used in the various computations.

Average width of bed B = 12.87 ft.

Representative grain diameter in the formula for rate of bed-load transportation = D_{35} = 0.25 mm. = 0.00082 ft.

Representative grain diameter in the roughness formula = D_{65} = 0.33 mm. = 0.00108 ft.

Average efficiency correction for the weighing tank = 1.273

Specific gravity of sediment = 2.65

Slope of banks = 1:1

Water temperature = 20° centigrade

Average water depth = d

Average cross-section area, A = d (12.87 + d)

Wetted perimeter of banks, P_w = 2.83 d

The various formulas, therefore, are

$$R_w = \left[\frac{V \cdot n_w}{1.486\ S^{1/2}}\right]^{3/2}$$

$$R_b = \frac{1}{12.87}\left\{A - R_w \cdot p_w\right\}$$

$$\Phi = \frac{Q_s}{12.87\ F g(\rho_s - \rho_f)}\sqrt{\frac{\rho_f}{\rho_s - \rho_f}}\ \frac{1}{g^{1/2}\ D_{35}^{3/2}} = \frac{Q_s}{300}$$

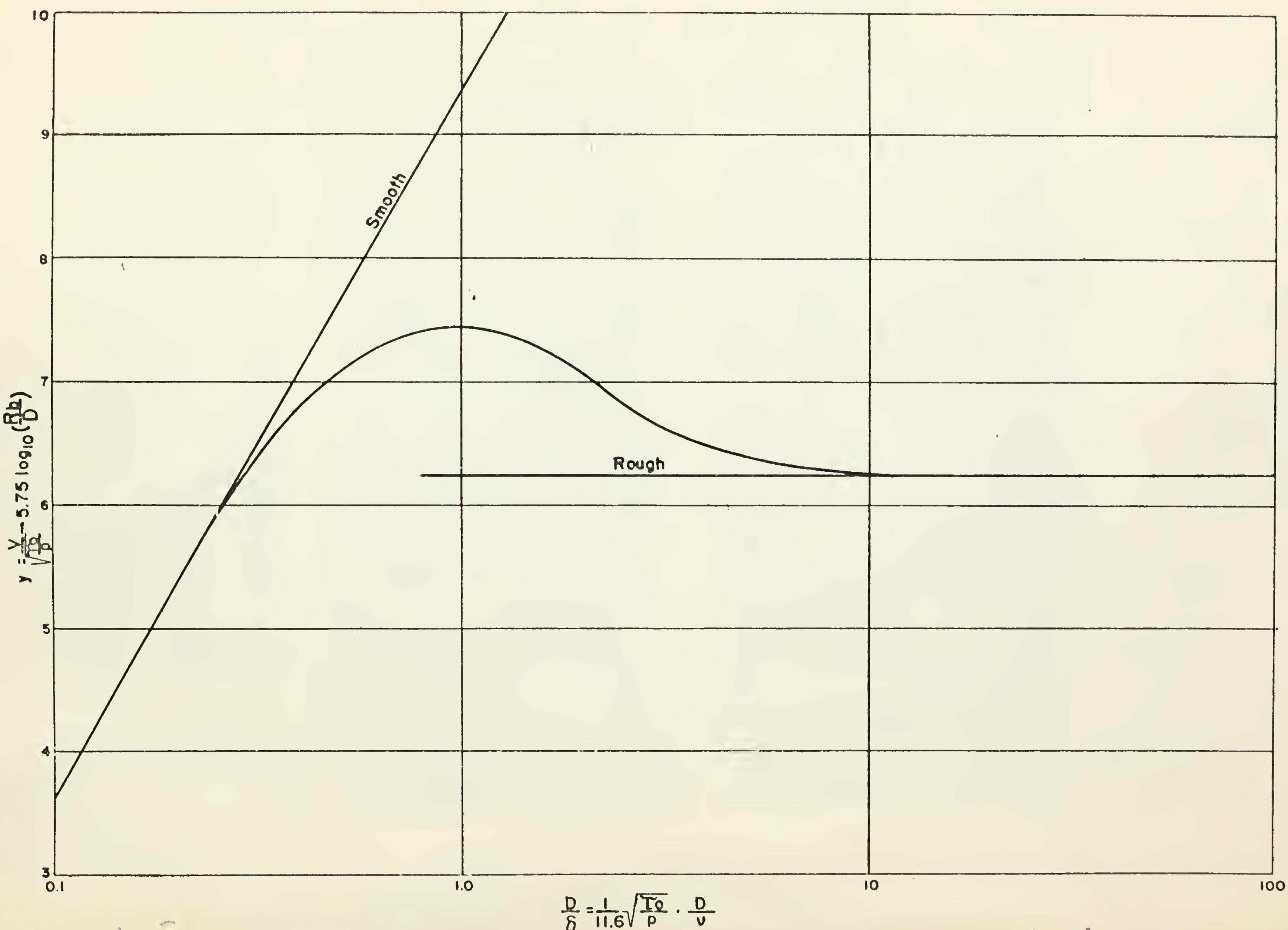

Figure 30—Curves for flow along smooth and rough walls.

$$\Psi = \frac{\rho_s - \rho_f}{\rho_f} \quad \frac{D_{35}}{S\ R_b}$$

$$\frac{D_{65}}{\delta} = \frac{D_{65}\sqrt{\tau_o/\rho}}{11.6\ \nu}$$

$$\Gamma = \frac{V/\sqrt{\tau_o/\rho}}{y + 5.75 \log_{10} (R_b/D_{65})}$$

Plots of the values of Ψ and Γ as a function of Φ for the various West Goose Creek measurements, table 8, are shown in figures 31 and 32. For comparative purposes the data from Mountain Creek are also shown in these figures. In the range of $\Phi > 1$ the Ψ points follow the curve derived for sands of uniform grain size much closer than do the measurements of Gilbert. In the experiments of Gilbert, however, it is important to note that part of the material caught in the collection tank possibly was really moving in suspension, and therefore, would not be included in the curve for transportation in single jumps of constant magnitude (3). The few suspended-load samples taken immediately below the hopper showed considerable concentrations in the higher layers of the flow while the usual characteristic increase in concentration near the stream bed was missing. The short duration of the floods, however, did not permit the taking of enough suspended load samples for a definite proof.

Examination of figures 31 and 32 shows an interesting trend of the correction factor, Γ , in the law for bed friction. This factor appears to decrease to a minimum value in the neighborhood of $\Phi = 1$ (maximum roughness) and then to increase again for larger values of Φ . A possible explanation for the shape of the curve is that for $\Phi > 0.01$ the transportation itself, and the accompanying change in the configuration of the bed surface increases the energy consumption of a particular flow. When Φ increases toward 1, another effect that appears to decrease the friction factor becomes important. The great number of bed particles in movement at any one time form a heavy and relatively slow moving layer between the bed and the flow proper. Newly developed eddies along the bed that are part of this heavy layer have considerable difficulty in rising into the higher layers of the flow. These eddies, because of their tendency to remain longer in the immediate vicinity of the bed, increase the scouring force of the flow (see $\Psi - \Phi$ curve).

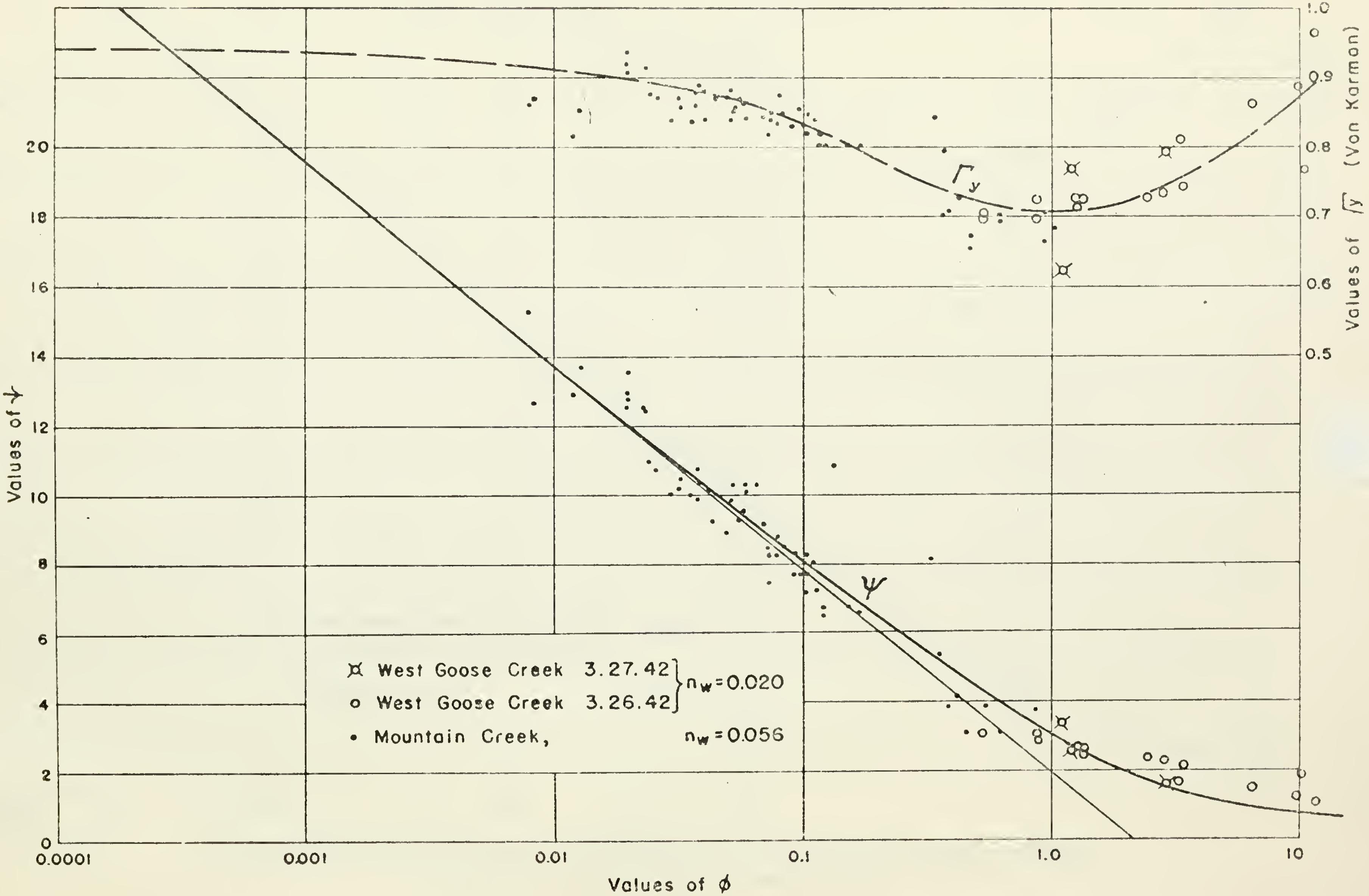

Figure 31–Comparison between data on bed-load transportation and bed roughness for Mountain Creek and West Goose Creek.

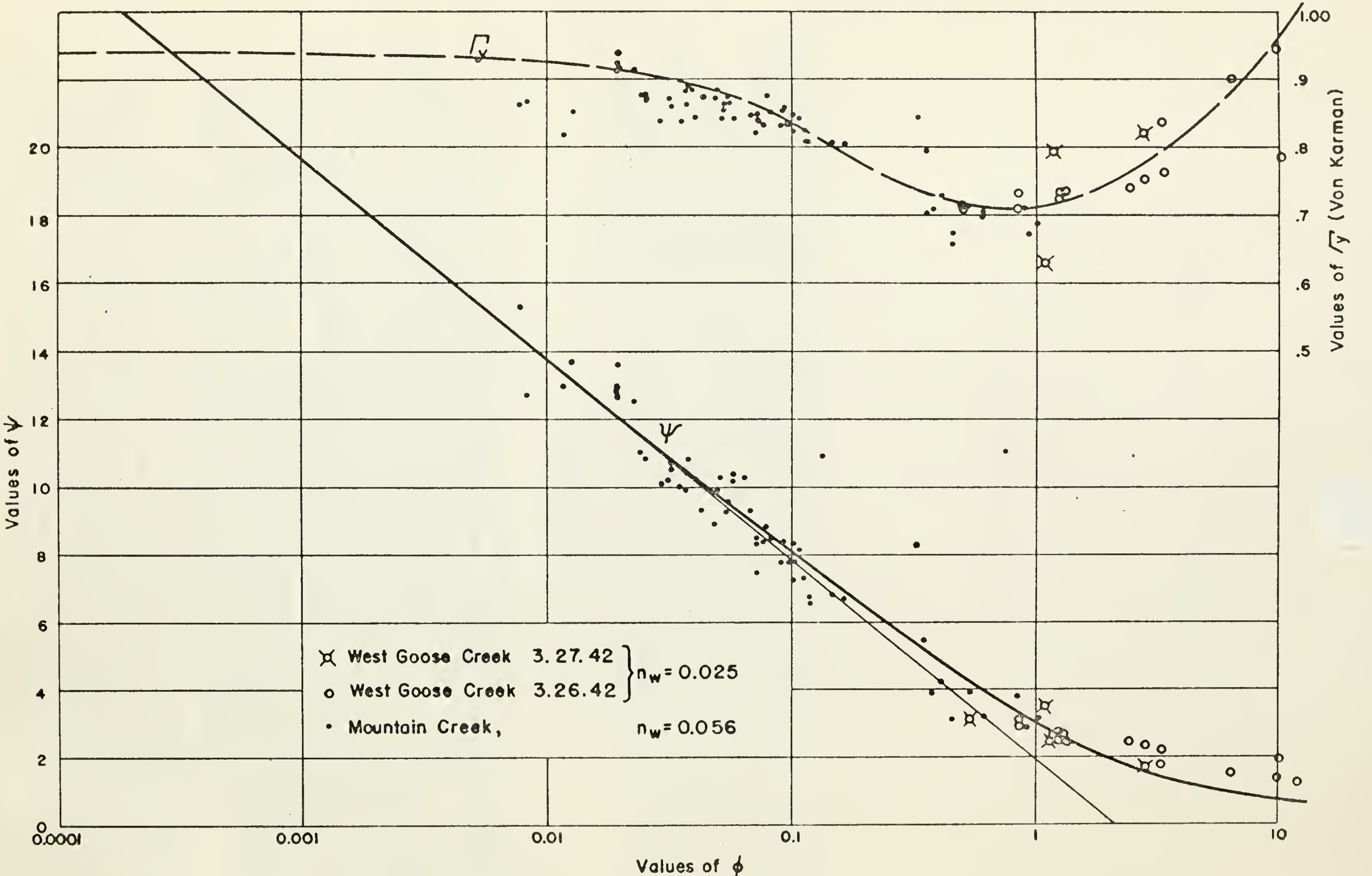

Figure 32—Comparison between data on bed-load transportation and bed roughness for Mountain Creek and West Goose Creek

When the eddies finally leave the vicinity of the bed they have lost more energy than would have been the case for clear flow and consequently are weaker. Their ability to cause "exchange of momentum" has decreased with the result that higher velocities are developed in the upper layers of the flow. The correctness of this reasoning can be checked only by additional measurements under a range of conditions comparable to those encountered in West Goose Creek.

A similar explanation has been used by Vanoni (15) to explain the increased stream velocity due to suspended load. He shows that within the turbulent exchange at any point of the cross section the concentration of upward-moving fluid elements is always higher than the concentration of downward-moving elements. Thus, energy is spent continuously lifting up that surplus concentration and is taken from the kinetic energy of turbulence, reducing its intensity faster than internal friction alone would account for. As in the case of bed load, the reduced exchange causes the average velocity to increase. The two cases differ mainly in the time when the reduction of turbulence occurs. Suspended load will decrease it <u>after</u> turbulence is distributed over the cross section while bed load will affect the newly-created turbulence <u>before</u> its distribution.

CONCLUSIONS

The measurements in West Goose Creek have extended the range of Φ from the previous value of 1 to the value of 10. If it can be assumed that this extension of the $\Psi - \Phi$ curve can be used for sediment that is coarser than found in West Goose Creek, it is possible to calculate the rate of bed-load transportation for rivers several feet deep and with sands approximately 1 mm. in diameter. There appears to be no reason why this assumption is not permissible. However, to prove definitely the validity of the assumption direct measurements are necessary in rivers of this size and sediment characteristics.

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List of Symbols Used

A	area (sq.ft.)
A_b	area pertaining to the bed (sq.ft.)
A_s	total annual (bed-load) transport (cu.ft./yr.)
A_t	total cross sectional area (sq.ft.)
A_w	area pertaining to the banks (sq.ft.)
B	width of the bed (ft.)
b	as subscript= pertaining to bed.
D	grain diameter or representative grain diameter (ft.)
D_{35}	sieve size that passes 35 percent by weight of a sand mixture (ft.)
D_{65}	sieve size that passes 65 percent by weight of a sand mixture (ft.)
d	water depth (ft.)
d_e	water depth in the end measuring point (ft.)
F	constant for settling velocity (3)
g	acceleration of gravity (ft./sec.^{2})
H_w	fall of the water surface between the ends of the measuring stretch. (ft).
n_b	Manning's roughness constant for the bed.
n_w	Manning's roughness constant for the banks.
P	wetted perimeter (ft.)
P_b	wetted perimeter pertaining to the bed (ft.)
P_w	wetted perimeter pertaining to the banks (ft.)
Q	water discharge (cfs)
Q_n	total mean runoff (cfs)
Q_s	total bed-load transport of a stream measured under water (lbs/hr.)
q_s	rate of bed-load transportation under water per ft of width (lbs/hr.ft).